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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

A THEORY OF SILT TRANSPORTATION

By W. M. Griffith,[1] Esq.

Synopsis

A theory of silt transportation is outlined in this paper, and equilibrium equations are presented which it is claimed are applicable to channels of all sizes and shapes, provided the silt load and the bed consist of "loose granular material," and provided certain hydraulic conditions are satisfied. The equations are of special value in problems relating to river-control works and tidal river outfalls; for example, they can be used to estimate the change in bed level that will result from widening or "tightening" a river section, or to determine whether a proposed dredge cut can be expected to maintain itself.

The study of silt transportation originated in the efforts of irrigation engineers to design artificial channels that would carry the heavily silt-laden waters of the rivers which supplied them, without silting their beds or scouring their banks. Attention is still mainly directed to this aspect of silt transportation, but the principles are actually of wider application. The same problem, in general, is involved in river-control works and in the improvement of the outfall of tidal rivers for navigation and drainage purposes.

The General Theory

In a paper[2] published in 1927, the writer advanced a theory of silt transportation, in which it was suggested that changes which occur in the cross-section of rivers and canals after artificial alteration of the existing conditions can be mathematically calculated by reference to a single elementary law. The theory was based on two assumptions: (1) That the transportation of silt is effected by the reaction of vertical eddies or the vertical resultant of other eddies; and (2) that the level of the bed at every point in a cross-section is governed by the law of silt transportation.

This theory is applicable to any channel section in which: (1) The bed of the channel is composed of loose granular material incapable of substantial

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by **September 15, 1938.**

[1] Cambridge, England.

[2] "A Theory of Silt and Scour," by W. M. Griffith, *Minutes of Proceedings*, Inst. C. E., Vol. 223 (1927).

shear resistance; (2) the silt load is of the same material; and (3) the flow is turbulent and follows the boundary surfaces.

The elementary law of silt transportation may be briefly stated thus: In a given cross-section of a stream, the condition for equilibrium at any point in the bed is given by the relation,

$$v = c\,d^n \qquad \text{(1)}$$

in which v is the mean velocity in the vertical through the point in question; d is the depth at that point; c is a coefficient varying with the amount of silt load carried; and n is an exponent the numerical value of which must slightly exceed 0.5.

Consider a naturally formed river section in which flow is taking place under the conditions stated (see Fig. 1). This may be segmental, semi-elliptical, parabolic, or of similar shape, but for simplicity's sake it will be designated parabolic. It is claimed that from Chezy's basic equations of flow for unerodible perimeters, the average velocities in the vertical planes through the points, A, B, C, etc., across the section will vary as the square root of the depths at those points; that is, as $d_1^{0.5}$, $d_2^{0.5}$, $d_3^{0.5}$, etc.[3] Now, if the silt load is uniformly distributed across the section, the velocities required for equilibrium under the law of silt transportation will vary as d^n. Hence, if the true value of n is 0.5, all points in the cross-section will be in equilibrium whatever its shape may be—deep and narrow, or broad and shallow; but if the value of n is either greater or less than 0.5, there can be only one value of d at which equilibrium is attained.

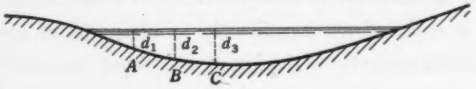

FIG. 1

Assume this value to occur at the mean depth of the parabolic section. Then, if the true value of n is less than 0.5, in the deep center the actual velocity will exceed that required for equilibrium by Equation (1) and in the shallow sides it will be less. The section, therefore, will tend to scour in the center and shoal near the sides and thus become deeper and narrower. If the true value of n is greater than 0.5, the reverse will be the case and the channel will tend to flatten out into a broad, shallow cross-section.

As channel sections carrying heavy silt loads do tend to flatten out, it is argued that the value of n must be greater than 0.5. However, observation shows that parabolic sections flowing under the conditions stated do have some stability; hence, the value of $(n - 0.5)$ cannot be great.

A parabolic section can be in equilibrium at all points if the silt load varies across the section directly as $\frac{v_m}{d^n}$, or, in other words, as $\frac{1}{d^{n-0.5}}$, since $\frac{v}{d^{0.5}}$ is constant across the section by the hydraulic law. If the value of $(n - 0.5)$ is small, this required variation in silt load across the section will not be great. By depositing silt where the actual velocity is in deficit, and picking it up by scour where the velocity is in excess, the stream will tend to adjust its silt load at different points across the section to that distribution which will give stability.

[3] *Minutes of Proceedings*, Inst. C. E., Vol. 223 (1927), pp. 245, 246, and 279.

Cross-eddies redistributing the silt charge will tend to upset this state of equilibrium. Hence, a channel of parabolic section, with a bed of sand and a silt charge of the same material, can have no permanent stability, and its temporary stability will depend on such absence of cross-eddies as can be gained by the channel being straight and of uniform section.

If a channel with an erodible bed is provided with smooth, unerodible sides, it can achieve stability only by flowing at one depth—that is, by forming a level bed across the section—and it invariably does so. A common example is found in the beds of irrigation canals which have to carry silt loads of sand in their upper reaches. An example will also be found in the beds of tidal rivers, either near their outfall, where the sides are lined, or at the outfall, where unerodible sides are provided by training walls.

Beyond the training walls, where no unerodible sides exist, any channel forming in the sand bed must of necessity have a parabolic section, but such channels have only a temporary stability. This stability is increased to some extent by the deposit, at the sides, of colloids, which produce a material having some shear resistance.

Development of Equations

It follows from Equation (1) that,

$$V_m = f\,c\,(d_m)^n \qquad (2)$$

in which V_m is the mean velocity of the whole cross-section and d_m is the mean depth; f is a factor depending on the shape of the cross-section. $f = \frac{V_m}{v_m}$, in which v_m is the mean velocity in the vertical through the point where the depth is d_m.

The value of f can be mathematically determined for any known type of cross-section. Thus, for a rectangular cross-section, $f = 1.0$; for a triangular cross-section, $f = 1.13$; and for a natural river cross-section,[4] its value will differ little from 1.1.

If B = the width at water surface, and Q = the discharge, then, $Q = V_m\,B\,d_m$, or,

$$V_m = \frac{Q}{B\,d_m} \qquad (3)$$

Substituting in Equation (3) the value of V_m from Equation (2),

$$\frac{Q}{f\,c} = B\,(d_m)^{n+1} \qquad (4)$$

Consider any two cross-sections of a channel, sufficiently close to each other to insure the value of Q and c to be unchanged. Let d_{m1} and d_{m2} be the mean depth of these two sections, and B_1 and B_2 their surface widths. Since for the same type of cross-section the value of f is constant, and since $\frac{Q}{f\,c}$ is constant,

[4] "A Theory of Silt and Scour," by W. M. Griffith, *Minutes of Proceedings*, Inst. C. E., Vol. 223 (1927), p. 253.

from Equation (4),

$$B_1\,(d_{m1})^{n+1} = B_2\,(d_{m2})^{n+1}$$

Therefore,

$$\frac{B_1}{B_2} = \left(\frac{d_{m2}}{d_{m1}}\right)^{n+1} \quad \text{(5)}$$

or,

$$d_{m2} = d_{m1}\left(\frac{B_1}{B_2}\right)^{\frac{1}{n+1}} \quad \text{(6a)}$$

Equation (6*a*) gives the values of changes in depths resulting from widening or tightening a river section when the types of the two cross-sections are similar.

Where the type of cross-section is changed, as for example from the parabolic to a rectangular type, as will occur at the site of a weir or bridge, the value of f will not be constant and Equation (6*a*) becomes,

$$d_{m2} = d_{m1}\left(\frac{f_1\,B_1}{f_2\,B_2}\right)^{\frac{1}{n+1}} \quad \text{(6b)}$$

in which f_1 and f_2 are the shape factors of the cross-sections at d_1 and d_2, respectively.

For a change from a natural river cross-section to a rectangular cross-section, the value of $\frac{f_1}{f_2}$ will approximate to 1.1.

In the fundamental equation (Equation (1)) since c varies with the silt load, it can be expressed in terms thereof. Let c_1 be the silt load of loose granular material (excluding colloids), expressed in terms of parts per 10 000 by weight, that is carried in the vertical above a point on the bed of a channel. Then,

$$c_1 = z\,\frac{v_m}{d^n} \quad \text{(7)}$$

in which z is a new coefficient and,

$$c_1 = z\,c \quad \text{(8)}$$

The term, "loose granular material," is intended to include shingle or sand down to an order of fineness of about $\frac{1}{500}$ in. in diameter, but it does not include light materials carried in suspension by all eddies—such as clay, organic matter, and sand of an extreme order of fineness. All such light materials, for the purpose of this paper, are classed as colloids because they do not fulfill the primary condition that their transportation is dependent on the vertical eddies alone.

If the value of $\frac{v_m}{d^n}$ is sufficiently high to transport the loose granular material, the colloids will remain in suspension and cannot be deposited, so they do not affect the bed. For this reason the beds of channels in which loose granular material is being transported will not be found to contain colloids in any quantity. Some quantity of fine sand, which by comparison with the loose granular material is of a second order of smallness, will be found within the interstices of the latter, where it finds shelter. However, the quantity will not

exceed the natural volume of the voids, and so will not affect the bulk of the bed material, or, consequently, the level of the bed at any point.

If the values of n and z are known, Equations (1), (2), (5), (6a), and (7) offer a mathematical solution to many river and canal problems.

The Value of n.—The value of n can be determined by a graphic analysis of the values of V_m and d_m of a series of channels of different depths, all carrying the same quantity of silt load and all having beds in equilibrium under this silt load.

This method was adopted by R. G. Kennedy in his classical paper.[5] However, Kennedy analyzed the data in terms of the maximum depth of the section, instead of the mean depth, because he made no fundamental assumption, and his analysis was purely for the purpose of discovering an empirical relationship applicable to a number of irrigation canals, all having more or less the same type of cross-section. A number of other observers followed the same process, but although the channels studied by any one observer may have been of a single type of cross-section, they were not necessarily similar to those studied by Kennedy or by any of the others.

Kennedy's value for the exponent, n, was 0.64, and other observers have arrived at values[6] varying from 0.44 to 0.727. As the beds of all the channels studied appear to have been of sand, such a large variation in the value of the exponent is illogical, because model experiments indicate that normal differences in the sizes of grains in the wetted perimeter do not materially affect the coefficient of roughness, or the conditions of flow.

Of course, if boulders are present in a river bed, they form definite obstructions to flow, of a different order of size, and the hydraulic conditions are altered. Their effect can be observed, for example, in time of flood, when the water surface will be found to be irregular, having the appearance of a series of waves, and indicating flow of a different order of turbulence. Hence, for shingle and boulder beds, a different value for n might be anticipated.

In his paper of 1927,[7] the writer found the value of n in Equation (2) to be equal to 0.59, approximately, by a graphic analysis of Kennedy's data in terms of d_m instead of d. In examples given in that paper, relating to sites on the Jumna and Ganges Rivers, which have beds of boulders and shingle, the writer assumed a value for n of 0.64. In 1930, a similar analysis of both Kennedy's data and the Madras[8] data showed that a value of n of 0.57 fitted both sets of data with reasonable accuracy.[9] Further investigation has led to the conclusion that this value may be accepted as approximately correct for beds of medium to fine sand. With regard to beds of shingle and boulders, a rough analysis of some flood discharges of the Jumna River gave a value of 0.65.

The Value of z.—Having now obtained a provisional value for n, the value of z in Equation (7) can be found from observations of silt content, velocities, and depths.

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[5] "Hydraulic Diagram for Canals in Earth," by R. G. Kennedy, *Minutes of Proceedings*, Inst. C. E., Vol. 119 (1895), p. 281.

[6] "Stable Channels in Erodible Material," by E. W. Lane, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 127.

[7] *Minutes of Proceedings*, Inst. C. E., Vol. 223 (1927), p. 279.

[8] "Critical Velocity Observations," Public Works Dept., Irrigation Branch, Madras, India, 1912.

[9] *Minutes of Proceedings*, Inst. C. E., Vol. 229 (1927), pp. 321, and 322.

An examination of the very complete collection of silt analysis and hydraulic data in a valuable paper by Harry F. Blaney, M. Am. Soc. C. E., and the late Samuel Fortier, M. Am. Soc. C. E.,[10] together with certain silt-sample analyses from the waters of the Wash at the tidal outfall of the River Great Ouse (which have been made under the writer's supervision), has led to the conclusion that a value of z of 7.5 in Equation (7) fits both sets of data with reasonable accuracy. This value is, therefore, provisionally recommended for graded sand (colloids being omitted); however, further investigation is required to determine how this value is affected by the size of the sand carried, and how far, if at all, it is affected by colloids if present in large quantities.

Substituting these values for n and z, Equations (1), (2), (6*a*), (6*b*), and (7) become, respectively,

$$v = c\, d^{0.57} \qquad (9)$$

$$V_m = f\, c\, (d_m)^{0.57} \qquad (10)$$

For similar types of section,

$$d_{m2} = d_{m1} \left(\frac{B_1}{B_2}\right)^{0.637} \qquad (11)$$

For dissimilar types of section,

$$d_{m2} = d_{m1} \left(\frac{f_1\, B_1}{f_2\, B_2}\right)^{0.637} \qquad (12)$$

and,

$$c_1 = \frac{7.5\, v}{d^{0.57}} \qquad (13)$$

Check of Equations with Observational Data

Canals.—The examination of silt analyses just referred to is given in Tables 1 to 4. Table 1 shows an analysis of the silt load in a cross-section of the Central Main Canal (Imperial Valley Canals, California). Columns (1), (2), (3), (4), and (6) are taken from Table 21 of the paper by Messrs. Fortier and Blaney,[10] and Columns (5), (7), (8), and (9) have been computed by the writer.

Column (7), Table 1, gives the arithmetic means of the four silt-load observations at each station. However, owing to the relatively high concentration of silt load near the bottom of the channel at certain stations, this arithmetic mean does not always give an accurate value of the true mean silt load. Hence, the observations at each station were plotted, as in Fig. 2, and the mean silt loads determined graphically from these plots are given in Column (8). It will be seen that where the relative value of the bottom silt load is high, the value of the arithmetic mean is much in excess of the mean value found graphically.

In Column (9), the values of $\frac{7.5\, v}{d^{0.57}}$ are entered, and a comparison of Columns (8) and (9) shows that a close agreement exists between the actual silt load and the load calculated from the hydraulic conditions by Equation (13).

[10] "Silt in the Colorado River and Its Relation to Irrigation," *Technical Bulletin No. 67* (1928), U. S. Dept. of Agriculture.

TABLE 1.—DISTRIBUTION OF SILT IN A CROSS-SECTION OF CENTRAL MAIN CANAL, IMPERIAL VALLEY CANALS

Distance between station and bank, in feet	DEPTHS, IN FEET		VELOCITIES, IN FEET PER SECOND		PROPORTION OF SILT BY WEIGHT RETAINED BY SIEVE No. 200, EXPRESSED IN PARTS PER 10 000			Values of $\frac{7.5\,v}{d^{0.57}}$
	Total (d)	To sampling point	At sampling point	v	Sample	Arithmetic mean	Graphic mean	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
6	5.3	0.5	2.57	2.36	2.2	6.02	5.8	7.0
		2.0	2.81		5.4			
		3.5	2.27		5.9			
		5.0	1.79		10.6			
10	5.8	0.5	3.01	3.11	1.6	7.6	8.6	8.6
		2.0	3.30		8.6			
		3.5	3.11		6.5			
		5.0	3.01		13.7			
14	5.1	0.5	3.50	3.29	1.7	8.25	7.1	9.8
		2.0	3.35		2.8			
		3.5	3.35		8.0			
		5.0	2.67		20.5			
22	4.7	0.5	3.55	3.18	2.6	12.1	10.3	9.8
		2.0	3.60		13.3			
		3.5	3.21		8.6			
		4.6	2.38		23.9			
26	4.8	0.5	3.35	3.21	1.8	16.7	9.1	9.94
		2.0	3.50		3.5			
		3.5	3.16		8.5			
		4.7	2.82		52.9			
30	4.8	0.5	2.82	2.92	1.1	14.4	8.8	8.9
		2.0	3.26		6.3			
		3.5	3.01		8.8			
		4.7	2.58		41.4			

Table 2 gives similar data for the Dahlia Canal, the observed data being taken from Table 22 of the Fortier and Blaney paper.[10] In this case, owing to the fact that only three silt samples were taken in the total depth, the graphic

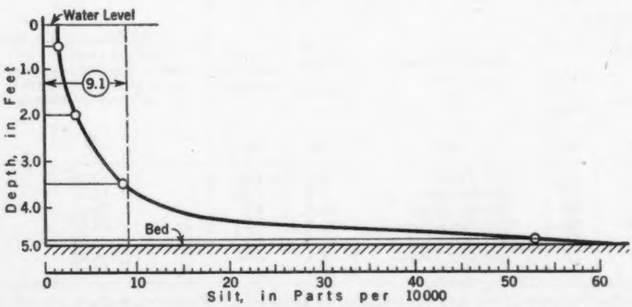

FIG. 2.—TYPICAL GRAPHIC DETERMINATION OF MEAN SILT LOAD

method of arriving at the mean silt content is not possible. Where the bottom values are relatively high, the arithmetic mean must be in excess of the true mean, which would account for the fact that the higher values in Column (7) are in excess of those in Column (8).

TABLE 2.—Distribution of Silt in Cross-Section of Dahlia Canal

Distance between station and bank, in feet	Depths, in Feet		Velocities, in Feet per Second		Proportion of Silt by Weight Retained by Sieve No. 200, Expressed in Parts per 10 000		Values of $\frac{7.5\,v}{d^{0.57}}$
	Total (d)	To sampling point	At sampling point	v	Sample	Arithmetic mean	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.5	1.6	0.20 0.96 1.40	1.34 1.39 1.05	1.26	1.1 5.3 21.1	9.2	7.3
4.5	1.2	0.20 0.72 1.00	1.74 1.98 1.54	1.75	1.3 13.3 50.0	21.5	11.9
7.5	1.2	0.20 0.70 1.00	2.13 2.08 1.59	1.93	1.5 9.1 72.8	27.8	13.2
10.5	1.2	0.20 0.72 1.00	1.64 1.98 1.69	1.77	0.7 11.1 40.2	17.3	12.0
13.5	1.0	0.20 0.60 0.80	1.44 1.49 1.34	1.42	1.2 4.5 30.8	12.2	10.65

Table 3 gives similar figures for other canals and for the Colorado River. In these cases the values of the bottom samples are not relatively high and the silt contents given by the arithmetic mean (Column (7)) would be more nearly correct. The very close agreement between the entries in Columns (6), covering a fairly wide range of depths and velocities, is somewhat remarkable.

TABLE 3.—Analysis of Silt Distribution in Various Canals and in the Colorado River at Yuma, Arizona

(Observations made on or near center vertical of cross-section)

Serial No.	Reference to *Technical Bulletin**		Name of canal or river	Total depth, d, in feet	v, in feet per second	Proportion of silt by weight retained by Sieve No. 300, expressed in parts per 10 000 (arithmetic mean)	Values of $\frac{7.5\,v}{d^{0.57}}$
	Table No.	Page					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	26	50	Ash Canal	3.2	3.22	12.1	12.38
2	27	51	Birch Canal	1.8	2.21	11.0	11.93
3	28	51	Alamo Canal	10.0	4.1	10.1	8.3
4	54	74	Colorado River	11.0	5.71	16.74	11.0
5	55	75	Alamo Canal	10.0	4.1	8.64	8.26
6	56	75	Brawley Canal	4.4	2.68	8.44	8.63

* "Silt in the Colorado River and Its Relation to Irrigation," *Technical Bulletin No. 67* (1928), U. S. Dept. of Agriculture.

Tidal River.—The Wash, at the outfall of the River Great Ouse, is shown in Fig. 3; Points *A*, *B*, and *C* indicate the stations at which observations were made. Series of observations at each of these points, covering various parts of the tidal cycle, are recorded in Table 4.

FIG. 3.—OUTFALL OF THE RIVER GREAT OUSE, SHOWING SAMPLING STATIONS

TABLE 4.—STUDY OF SILT DISTRIBUTION AT TIDAL OUTFALL OF RIVER GREAT OUSE

Sample No.	Depths, in Feet and Inches		Surface velocity, v_s, in feet per second	Silt Content, in Parts per 10 000		Value of $\frac{7.5 v_s}{d^{0.57}}$
	To sampling point	Total		Sample	Mean	
	(a) Observations at Site *B*, Taken During an Average Neap Tide, from Low Water to High Water					
1 2	3–0 7–0	10–0	0	1.23 1.14	1.18	0
3 4	3–0 6–0	9–0	0.31	1.05 1.37	1.21	0.663
5 6	3–0 6–0	9–6	1.5	0.88 1.48	1.18	3.12
7 8	3–0 6–0	9–0	1.7	2.72 3.92	3.32	3.64
9 10	7–6 3–6	11–3	1.9	3.54 3.48	3.51	3.53
11 12	7–6 4–9	11–6	2.23	4.22 3.62	3.92	4.14
13 14	4–8 8–0	11–0	2.33	3.06 5.0	4.01	4.43
15 16	4–0 8–0	12–0	2.3	3.95 4.58	4.26	4.18
17 18	5–6 11–0	16–0	3.0	3.89 3.58	3.73	4.58
19 20	12–0 6–0	17–6	1.88	3.24 2.48	2.86	2.74
21 22	7–0 14–0	21–0	1.47	2.35 2.61	2.48	1.93
23 24	7–6 14–9	22–0	1.30	1.43 1.575	1.5	1.66
25 26	8–0 16–0	24–0	0.70	1.43 1.19	1.31	0.855
27 28	7–10 15–10	23–6	0.40	.... 1.13	1.33	0.494

TABLE 4.—(*Continued*)

Sample No.	Depths, in Feet and Inches: To sampling point	Depths, in Feet and Inches: Total	Surface velocity, v_s, in feet per second	Silt Content, in Parts per 10 000: Sample	Silt Content, in Parts per 10 000: Mean	Value of $\frac{7.5\,v_s}{d^{0.57}}$
(*b*) Observations at Site *B*, Taken During a Low-Spring Tide, on the Latter Part of the Ebb						
61 62	5–0 10–0	15–0	2.21	2.4 7.6	5.0	4.95
63 64	3–8 7–4	11–0	2.80	2.6 10.7	6.65	7.7
65 66	3–0 6–0	9–0	2.6	8.32 4.94	6.63	6.8
69 70 71	Surface 3–0 6–0	6–6	1.8	4.65 6.30 10.90	6.28	7.5
72 73 74	Surface 3–6 6–6	7–0	1.1	4.43 4.67 4.45	4.48	4.68
75 76 77	Surface 3–6 7–0	7–6	0.8	5.28 5.58 4.93	5.26	3.66
78 79 80	Surface 3–9 7–0	7–6	Nil	2.66 3.42 4.78	3.62	Nil
(*c*) Observations at Site *A*, Taken During a Low Spring Tide, on the Ebb (Rough)						
81 82 83	8–0 16–6 24–0	25–0	2.93	3.39 1.93 1.92	2.41	3.51
84 85 86 87	Surface 21–6 14–0 7–0	22–0	2.65	1.70 3.60 3.58 2.35	3.74	3.40
88 89 90 91	Surface 21–6 14–6 7–3	22–0	3.22	2.31 4.20 3.59 3.10	3.3	4.10
92 93 94 95	17–6 Surface 6–0 12–0	81–0	3.20	3.57 3.24 3.80 4.02	3.66	4.45
96 97 98 99	Surface 15–6 5–3 10–6	16–0	3.56	3.80 5.85 4.92 4.92	4.87	5.44
100 101 102	4–8 13–6 10–4	....	4.16	4.87 6.92 6.20	6.0	6.94

The velocities in Table 4 are surface velocities. Observation indicates that the relationship between the surface velocity, v_s, and the mean velocity in a vertical plane, v, agrees roughly with Bazin's relationship between the maximum surface velocity and the mean velocity of the entire section; that is, the ratio of v_s to v is on the order of 1.28 to 1. On the other hand, the silt contents given

TABLE 4.—(*Continued*)

Sample No.	Depths, in Feet and Inches		Surface velocity, v_s, in feet per second	Silt Content, in Parts per 10 000		Value of $\frac{7.5\, v_s}{d^{0.57}}$
	To sampling point	Total		Sample	Mean	
(*d*) Observations at Site *C*, Taken During a Neap Tide, from Low Water to High Water						
30	3–0	6–0	0	1.63	1.63	0
31 32	2–2 4–4	6–6	1.55	1.93 1.55	1.74	0.26
33 34	2–9 5–6	7–3	0.33	2.75 2.80	2.77	0.8
35 36	2–10 5–8	8–6	0.43	2.23 2.00	2.11	0.96
37 38	5–2 2–7	7–9	0.533	2.60 2.58	2.59	0.776
39 40	5–6 2–9	8–3	0.46	2.61 2.45	2.53	1.03
41 42	6–6 3–3	10–0	0.87	2.20 1.82	2.01	1.78
43 44	3–8 7–4	11–0	1.43	1.93 2.83	2.38	2.72
45 46	4–8 9–4	14–0	1.66	2.56 3.03	2.80	2.77
47 48	5–8 11–4	17–0	1.7	2.67 3.55	2.86	2.5
49 50	6–0 12–0	18–0	1.6	(?) 3.04	....	2.05
51 52	6–8 13–4	20–0	1.13	2.60 2.85	2.72	1.52
55 54	7–0 14–0	21–0	0.7	2.68 2.73	2.70	0.916
55 56	7–0 14–0	21–0	0.51	2.73 2.76	2.74	0.665
57 58	6–10 13–3	20–6	0	1.45 2.27	1.86	0
59 60	6–10 13–8	20–6	0	2.37 2.29	2.33	0

in Table 4 include colloids, because apparatus for the separation of the colloid portion was not available.

Owing to the tendency of salt water to deposit colloids, the colloid content in these samples was not great. However, it must be greater on the ebb tide, or outflow, than on the flood tide, or inflow. It will be less, the farther out from Point *A* (Fig. 3), the observation point is and will be small at Site *C*.

Fig. 4 shows the variation in silt content at Site *B* on a neap tide, during the period from low water to high water, throughout which the velocities and depths were changing. (These observations were taken from a boat at anchor and the length of anchor chain permitted of some change of position under the varying flow, so the samples were not all taken at exactly the same spot.)

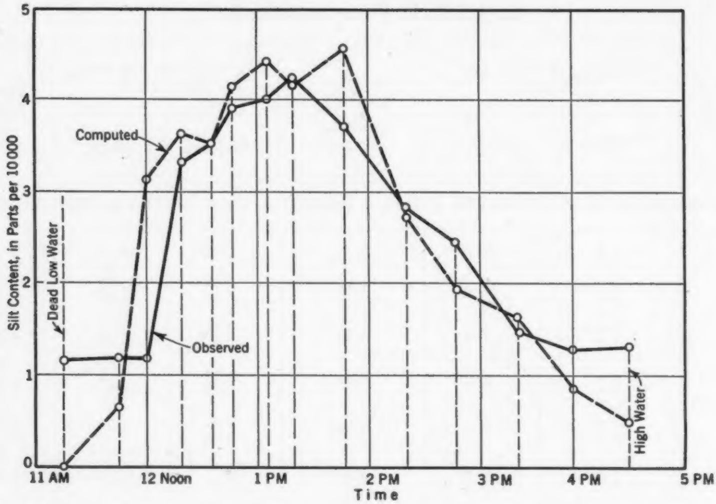

FIG. 4.—SILT CONTENT AT SITE *B*; AVERAGE NEAP TIDE, LOW WATER TO HIGH WATER

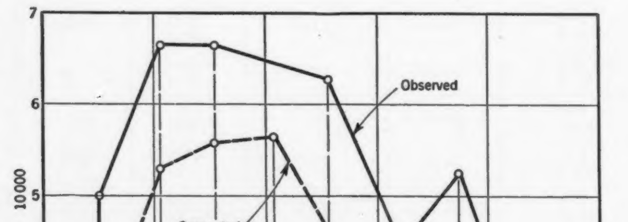

FIG. 5.—SILT CONTENT AT SITE *B*; LOW SPRING TIDE, EBB

The curve of values of $\frac{7.5\, v_s}{d^{0.57}}$ is also plotted. It will be noticed that the two curves show a close agreement, but that there is a time lag of about 30 min between them, as might be expected. Details of these observations are given in Table 4(*a*).

Fig. 5 gives similar curves at the same site for an ebb tide. Again, the curves are similar, but in this case the observed silt content is continuously in excess of that computed by formula. This fact is presumably due to the increased quantity of colloids on the ebb flow. The values are tabulated in Table 4(*b*).

Fig. 6 and Table 4(*c*) give observations taken for part of the ebb flow at Site *A*. In this case, despite the fact that owing to the ebb flow the colloid content would be relatively great, the computed curve lies above the curve of observed silt content. In explanation it may be stated that at Site *A* conditions are changing so rapidly that the actual silt content lags behind that computed from the hydraulic conditions.

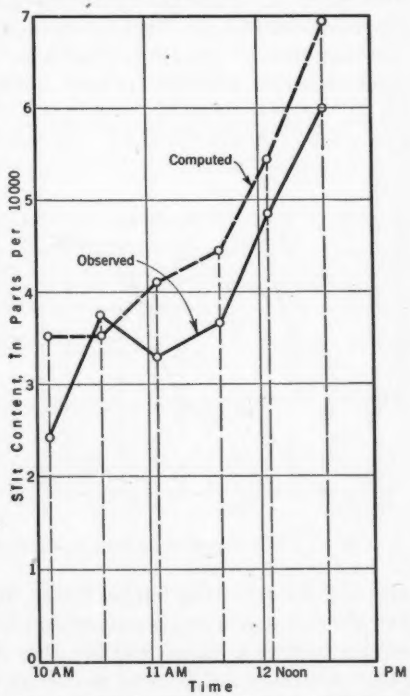

FIG. 6.—SILT CONTENT AT SITE *A*; LOW SPRING TIDE, EBB (PART ONLY)

Fig. 7 and Table 4(*d*) give similar curves for a neap tide, from low water to high water at Site *C*. At that site, velocities are lower and depths greater, and there is a considerable increase in wave action. This wave action, in the shallow waters of the Wash, creates a turbulence sufficient to maintain a considerable silt content even when the velocity is zero. Under such conditions, the hydraulic factors have little effect on the silt content.

The turbulence created by wave action depends on the height of the waves, which, in turn, depends on the area of the water surface. This area is greater at high tide than at low tide on account of the area of sand banks then exposed, and explains why the silt content is greater at high water than at low.

APPLICATION OF THEORY

Canals.—In canal systems required to carry quantities of graded sand together with colloids as silt loads, the conditions named apply in the upper reaches. In the lower reaches much of this graded sand may have passed out of the system and a different condition may pertain.

The difference between these two conditions may be seen from the difference in the type of cross-section. In the upper reaches the section will be nearly

rectangular, the bed being approximately level except close to the sides, where the reduction of velocity resulting from the horizontal eddies from the sides will cause a slight rise of bed. In the lower reaches, if the silt load consists of light material only, the bed will be curved.

In turbulent flow within stream-lined boundaries the eddies in general are a function of R, the hydraulic radius. However, the vertical eddies (or the vertical resultants of the eddies) are a function[11] of d_m. Therefore, if the silt content is largely colloidal, the silt transporting power will be a function of R^n instead of $(d_m)^n$, and the lighter the material the more closely will the value of n approximate to 0.5. Under such conditions the theory outlined in this paper is not applicable. However, the light material found in the lower reaches of canal systems usually deposits slowly, and when excavated forms a useful

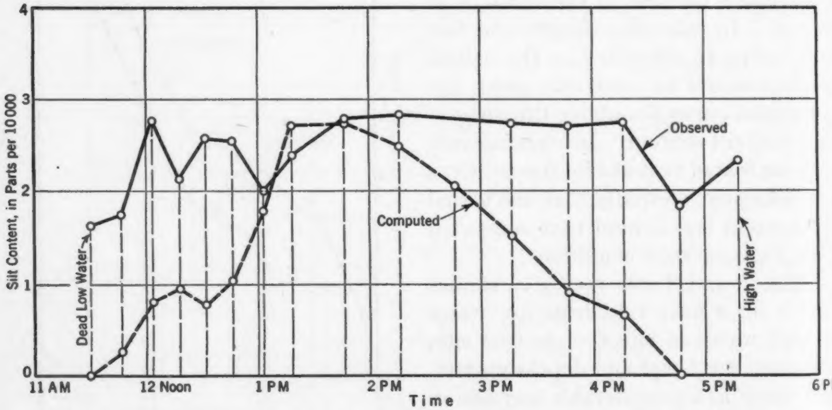

FIG. 7.—SILT CONTENT AT SITE *C*; AVERAGE NEAP TIDE, LOW WATER TO HIGH WATER

material for repairing banks; hence, it is often more beneficial than harmful, and the problem of its transportation is usually not a serious one. The presence of clay insures binding, and the very fineness of the sand appears to give it a shear resistance not present in coarser sand.

Where the present theory is applicable, the condition required to insure that the full silt load of the parent stream will be carried by the offtake canal is that the value of $\frac{V_m}{(d_m)^n}\left(\text{or, for ordinary silt, } \frac{V_m}{(d_m)^{0.57}}\right)$ of this offtake canal shall not be less than the value of this ratio in the parent channel. The question of whether the inflow to the canal actually has a higher or a lower silt concentration than the parent channel itself, depends on the hydraulic conditions at the offtake. By scientific design of the offtake, it is possible to reduce the silt load greatly in the offtake channel.

Again, where the silt charge is heavy, and the surface slope limited, it is desirable to provide a canal section having the maximum silt-carrying capacity. Such a section is secured if the ratio $\frac{V_m}{(d_m)^n}$ has a maximum value,

[11] *Minutes of Proceedings*, Inst. C. E., Vol. 223 (1927), pp. 276 and 277.

and the width and depth of bed giving the maximum value for this ratio can be found by plotting its values for different types of sections.[12]

In the foregoing it has been assumed that the most suitable side slopes for the channel have been predetermined. In practice, the correct value for the side slopes is found by experience, and will depend on such factors as: (1) Vegetation at the sides; (2) amount of colloids in the silt content; (3) angle of repose of the natural material; (4) velocity; and (5) ruling radius of curvature.

It is not claimed that this or any theory of silt transportation can solve all silt difficulties of canals. The writer's experience indicates that most cases of silt trouble are most easily and cheaply dealt with by reducing the silt content at the head of the canal by a more scientific design of the intake works. The writer considers that Kennedy's original theory, by concentrating so much attention on the detail of the channel design, has diverted attention from the study of the silt distribution at the offtake, where the true solution to the silting problem may usually be found. The Imperial Valley Canals appear to be a case in point.

River Control Works and Tidal River Outfalls.—As already stated, Equations (6*a*), (6*b*), or Equations (11) or (12), provide a means of determining changes in bed level which may be anticipated from widening or "tightening" existing river sections. An example is afforded by the three cross-sections of the Hundred Foot River given in Fig. 8. Approximately $\frac{5}{8}$ mile of this river, between Cross-

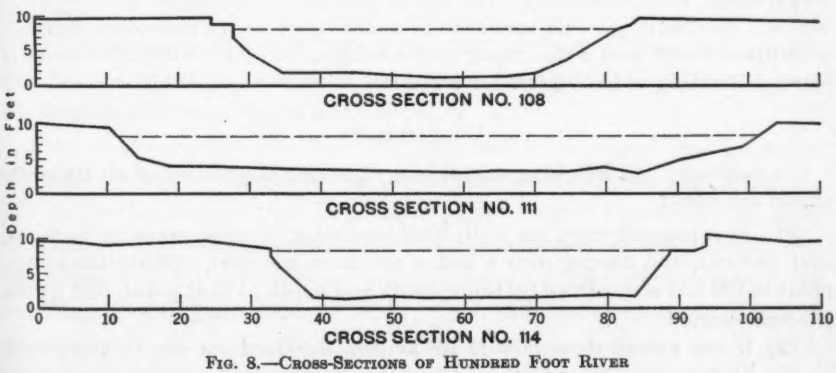

FIG. 8.—CROSS-SECTIONS OF HUNDRED FOOT RIVER

Section No. 108 and Cross-Section No. 114, was widened ill advisedly. Cross-Section No. 111 is a typical cross-section of this widening. The elevation of the water surface varies, but the mean water level is about as shown in Fig. 8. The necessary hydraulic data for testing Equation (11) are given in Table 5.

In Equation (11), let B_1 and d_{m1} be the mean values of B and d_m, respectively, for the unwidened sections, and let B_2 be the width of the widened section. Then, $d_{m2} = d_{m1}\left(\frac{B_1}{B_2}\right)^{0.637} = 5.725\left(\frac{54.9}{89.8}\right)^{0.637} = 4.18$ ft, the calculated value of the mean depth of the widened section. This value compares with an observed mean depth of 4.08 ft, a difference of less than 3 per cent.

[12] *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 184 and 185.

As a general example of the use of this theory in tidal river outfall problems, mention again may be made of the River Great Ouse. In recent years the tidal section of that river has silted sufficiently to threaten the safety of the low-lying fen country which it drains. Several types of remedial measures have been proposed, but opinions have differed as to their respective merits.

TABLE 5.—HYDRAULIC DATA FOR HUNDRED FOOT RIVER

Cross-Section No.	B, in feet	Area, in square feet	d_m, in feet	Remarks
108	53.8	304	5.65	Unwidened sections
114	56.0	325	5.80	Unwidened sections
...	54.9	...	5.725	Mean value of unwidened section
iii	89.8	366	4.08	Widened section

By the use of the present theory, a study of the hydraulic conditions of the non-tidal and tidal sections of the River Great Ouse and its tributaries enables one to trace the true cause of recent changes in bed level and the actual reason for the present silting trouble. Further, the theory enables one to calculate, within limits, the changes in bed which may be anticipated to result from each of the various outfall improvement projects that have been proposed. Finally, it shows the inadvisability of dredging a deeper channel without altering the hydraulic conditions to insure that the greater depth would be hydraulically maintained; and, conversely, that if the hydraulic conditions were properly altered, the dredging would appear to be unnecessary, as the improved hydraulic conditions would insure the required alteration in bed level within the range of strata consisting of loose granular material.

CONCLUSIONS

In summary, the following conclusions regarding this theory of silt transportation are noted:

(1) In a channel carrying a silt load consisting of loose granular material, such as sand, and flowing over a bed of the same material, equilibrium at any point in the bed is governed by the velocity and depth at that point, and by the silt load carried.

(2) If any two of these factors are known, the third one can be computed.

(3) Statements (1) and (2) are true for the channel section as a whole, equilibrium being governed by the mean velocity, mean depth, and average silt load across the section.

(4) A channel flowing under these conditions will tend to flow at one uniform depth across the section, the depth which gives equilibrium independent of the volume of flow.

(5) In the case of a channel carrying only very light material (capable of being carried by all eddies, and classifiable as "colloids") and flowing over a bed of the same material, different conditions pertain and the type of cross-section is then different.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

WATER-SOFTENING PLANT DESIGN

By W. H. Knox,[1] M. Am. Soc. C. E.

Synopsis

In Ohio, the water supplies available in sufficient quantities to justify their development for municipalities are hard, almost without exception. The ground-waters used for municipal supplies range from about 60 ppm at a few supplies, derived from the sandstone region in the southeastern part of the State, to about 2 000 ppm in the northwestern part. The hardness of the surface water supplies does not vary through such a wide range. The average hardness of the Ohio River water throughout the year is about 100 ppm and the average hardness of the Lake Erie water is about 115 ppm. Most of the surface waters from streams within the State have an average hardness between 100 and 400 ppm; the maximum and minimum hardness of these surface waters fluctuates considerably above and below these averages.

Thus, it is seen that Ohio has been, and still is, a natural field for water softening. The first water-softening plant in the United States was installed in 1903 at Oberlin, and the first large plant in the United States was placed in operation at Columbus, in 1908. Very few plants were installed during the next fifteen years, there being a total of only six municipal water-softening plants installed in Ohio prior to 1924. Since 1924 the number of municipal water-softening plants has increased rapidly and, when plants now (April 1, 1938) under construction are completed, there will be 93 plants in the State. A total of approximately 4 800 000 population is supplied by the 450 municipal water-works systems in Ohio and 1 450 000 of these persons are supplied with softened water.

It is required by State statutes that the plans for water-works improvements (including water-treatment plants) shall be submitted to, and shall have received the approval of, the State Department of Health before construction of such works is undertaken. In reviewing the plans for such works since 1924,

Note.—This paper was presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 21, 1937. Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by **September 15, 1938.**

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the writer has had an excellent opportunity to observe the trend of the design of water-treatment plants in the State. This experience has been supplemented by visits to plants to observe the results being obtained and by the summaries of daily reports of operation that are submitted to the Department monthly. The size of the existing plants varies from the smallest plants, which have capacities of only 8 000 to 10 000 gal daily, to the Cincinnati Plant which has a rated capacity of 160 mgd. The plants in Ohio represent almost every type that can be constructed.

In this paper, an attempt has been made to avoid discussion of plant operation, the chemistry of the processes, and comments on patented equipment, except in so far as these items do affect the design of a plant.

Although the first water-softening plants were installed almost as early as the first rapid sand filtration plants, many more of the latter were built during the early years of the Twentieth Century. As a result, the design of the earlier water-softening plants was patterned somewhat after the design of water-filtration plants. The defects of such design and the need of special features for water softening were discovered by the operation of the plants and, hence, there has been a gradual change in design so that at present a softening plant is distinctly different from a filtration plant in which softening is not a part of the process. In this paper, only the special features of water-softening plants, which are not also features of filtration plants, will be emphasized.

Source of Supply

If an existing well-water supply has a hardness of more than 700 to 800 ppm, another source of supply should be sought if a water-softening plant is to be installed. The cost of chemicals for water softening is, roughly, 1 cent per 1 000 gal for every 100 ppm the hardness is reduced. Hence, the cost of softening becomes burdensome when a very hard water is to be treated.

Aeration

For surface waters, aeration is sometimes an assistance in the removal of tastes and odors. For ground-waters it is advisable if considerable carbon dioxide or sulfur occurs in the water.

In recent years, patented aerators have been introduced as substitutes for the spray nozzles, coke trays, and step aerators. Limited tests have shown that such patented aerators require just as much head as, and are not any more efficient for aeration or removal of carbon dioxide than, non-patented devices. However, the patented devices are much more sightly, occupy less space, and cause less splash than non-patented devices. Therefore, consideration should be given to patented devices if their cost is reasonable.

Provisions should be made so that the aeration devices can be by-passed, the aerator devices cleaned, and the cleaning water drained to waste.

Lime-Soda Water Softening

The devices for handling chemicals—for adding chemicals to the water and for mixing them—are of major importance at a lime-soda softening plant. These features should be considered first and the remainder of the plant built around these units.

Chemical Feed Devices.—Dry-feed machines have almost entirely superseded solution-feed devices for such plants except in small municipalities that have industrial or railroad types of softening plants, in which solution-feed devices are generally used.

In small plants it is desirable to provide an extra machine that can be used for more than one chemical; at larger plants, duplicate machines should be provided for each chemical. The machines should be placed as close as possible to the point of application so that the chemical solution can flow by gravity from the machines through short hose lines to the points of application. If this is not feasible, the chemical solutions should be conveyed in an open trough, or in hose lines supported continuously. If pipes are used to carry chemical solutions, such pipes should be accessible and should be provided with unions or other devices so that they can be removed and replaced quickly. Crosses at changes in direction to permit hose connections for flushing will assist in keeping the lines used for some chemicals clean; for lime, such flushing is useless.

Pneumatic equipment should be used for handling chemicals at large softening plants if the chemicals must be elevated. This is the most economical method of handling large quantities and permits the receipt of chemicals in bulk, which reduces the cost. For the plant of medium size a separate second-story room should be provided for the storage of chemicals in bags. Provisions should be made for elevating them to the storage room either by a hoist or by a bag conveyor. They can be dumped through openings in the floor of the storage room to the hoppers of the dry-feed machines. It is essential that connections to the hoppers be dust-proof.

For very small plants it is satisfactory to store chemicals in a room adjacent to the dry-feed machines and to carry the supply to the hoppers as needed.

Quick or Flash-Mixing Devices.—At extremely large plants it is desirable to obtain an intimate and thorough mixing of the chemicals with the water to insure that the water passing to all the subsequent treatment devices shall be of uniform quality. The hydraulic pump is used in a patented process as a flash mix at the Baldwin Water Purification Plant, at Cleveland, Ohio, and at the water-softening plant at Cincinnati. Various mechanical patented devices will also produce a flash mix. They have also been installed at some moderately large plants in Ohio. The writer questions the need of flash-mixing devices at medium or small plants, since a thorough mixing is obtained so quickly in the mechanical mixing devices at a water-softening plant.

Mixing Devices.—Baffled mixing chambers were used at the earlier plants. Mechanical mixing devices are preferable and have been installed at most of the plants constructed since 1924. Prolonged mixing is necessary to secure as complete reaction as possible and to avoid waste of chemicals. A detention period of 30 to 40 min is desirable and in no case should the detention period be less than 20 min.

A wide variety of stirring devices have given fairly satisfactory results. In many of the earlier plants, they consisted of horizontal or vertical paddles rotated through a vertical shaft by a motor on top of the mixing chamber. Recent trends are toward patented mixers. The devices should provide velocities between 1 and 1.5 ft per sec.

When the plant is large enough to justify the installation of two mixing chambers, it is desirable to have the two basins arranged so that they can be operated either in series or in parallel. It is necessary, of course, to have provisions for draining the chambers completely, preferably by gravity. Large-sized hose connections should be provided for flushing sludge from mixing chambers, clarifiers, and settling basins.

Clarifiers.—The quantities of sludge produced in water-softening plants are much greater than at plain purification plants. Unless provisions are made for frequent removal, the capacity of the settling basins is soon decreased materially by accumulated sludge, in which case storage capacity must be provided in the design. For this reason, and also due to the fact that, in many instances, small quantities of sludge can be disposed of more easily than accumulated quantities, the settling basins at most water-softening plants are equipped with devices for continuous or frequent removal of sludge.

The settling basins at a water-softening plant (and also at a filtration plant) should be true settling basins rather than coagulation basins; that is, the chemical reactions and coagulation should be practically completed before the water enters the settling basins. This item, together with the fact that the precipitate in lime softening is heavier and settles rapidly, permits much shorter detention periods for settling basins equipped with devices for sludge removal than the long detention periods formerly provided for coagulation basins at water-purification plants.

Either a rectangular basin with a longitudinal flow, or a circular basin with radial flow, gives satisfactory results. Square basins with transverse flow tend to short circuit and, as a result, no basins of this type have been installed recently.

Most engineers are guided, to some extent at least, by the typical layouts furnished by the manufacturers of the sludge-removal equipment which they propose to use. Competition among manufacturers can be obtained on bids for sludge-removal equipment for either rectangular or circular basins. There is one item that must not be overlooked in the design of a plant where clarifiers are used: In most clarifiers the effluent discharges over a weir, and this causes a practically constant water level in the clarifiers; in most coagulation basins the effluent is drawn off through submerged orifices, or over submerged weirs, which permits a considerable fluctuation of the water level in the basins. Such fluctuation is not provided when the clarifier effluent discharges over a weir. Hence, the only storage of water available for filtration is that in the carbonation chamber and in the filters themselves, above the level of the wash-water troughs. This storage is quite limited and may not be sufficient to provide the necessary "cushion" between the low-service and the high-service pumps. Lack of such storage may cause difficulties in operation unless automatic devices are installed to secure co-ordination of the various devices. This

"cushion" or storage may be secured in clarifiers by causing the effluent to discharge through submerged ports which will permit the water elevations to fluctuate.

If clarifiers alone are used as settling basins, a detention period of 2 hr is generally sufficient to secure a satisfactory clarified effluent. Some of the chemical reactions are not entirely completed within this period; nor would they be in several times this detention period. This deficiency is partly taken care of by recarbonation of the water.

It is essential that provisions be made for completely draining the clarifiers, preferably by gravity, in order to secure access to the sludge-removal mechanism. In colder latitudes a cover or superstructure over the clarifier may be necessary to avoid ice troubles, especially if a surface water is to be treated.

Plain Settling Basins.—At small plants it may still be desirable for various reasons, to install settling basins without sludge-removal equipment. Such basins should be designed and operated in a manner similar to the coagulation basins of a purification plant. A detention period of 4 to 6 hr should be provided for such basins.

Efforts to withdraw sludge through a manifold system or series of small openings in the floor of such basins while the basins are filled with water have been unsuccessful at two Ohio plants; such withdrawal of sludge can be, and is being, accomplished at industrial types of water-softening plants when the depth of water is at least as great as the diameter of the tank. Since sludge from plain settling basins is generally disposed of by discharging it into streams during high flows, it is essential that the basin be set sufficiently above flood levels so that it can be completely drained into the stream during flood periods.

Sludge Disposal.—This is one of the chief difficulties of lime-soda water softening. The disposal of lime sludge into a stream does not constitute "pollution" such as that caused by the direct discharge of sewage, but it does cause unsightly deposits on the banks and bed of the stream. However, this is still the most common method of sludge disposal; it is satisfactory only when ample flows and velocities in the receiving stream are available. One Ohio plant is fortunate in having a near-by abandoned quarry into which sludge can be discharged. Another large plant discharges the sludge into up-ground lagoons surrounded by earth embankments. Other plants discharge the sludge alternately into two or more lagoons, allowing one lagoon to dry out while the other one is in use; at these plants some of the accumulated sludge is sold or is given away to farmers for use as "agricultural lime." The latter method is fairly satisfactory when the sludge produced can be absorbed by local demands. Limited experiments have been made on sludge drying by drum filters. Experiments are also being made of incinerating the sludge to convert it to calcium oxide so that it can be used again in the water-softening process.

In sewage treatment, due to the offensive nature of the sludge produced, the disposal problem had to be solved, and the engineers and equipment manufacturers worked on this problem until, at present (1938), satisfactory sludge disposal can be provided for any sewage plant. A satisfactory means of disposing of lime sludge must be found within the near future, or a lime-soda type of water-softening plant will often not even be considered for this reason alone.

Recarbonation of Water.—Lime softening produces a supersaturation of normal carbonates of calcium and magnesium in the water. In order to avoid delivering an incrusting water, it is necessary to convert the normal carbonates into bicarbonates, and this is accomplished by applying carbon dioxide to the lime-softened water.

Before passing to the filters, the effluent from the clarifiers or settling basins is passed through a chamber in which carbon dioxide is applied to the water. On account of the high solubility of carbon dioxide gas, especially in caustic water, only a brief detention period is required. For structural reasons such chambers are generally compartments adjacent to, and of the same depth as, the settling basins. Although the reactions are practically instantaneous, a detention period of 5 to 10 min, or even a longer period, is generally provided.

Carbon dioxide is applied through grids of perforated pipes usually laid on, or slightly above, the floor of the chamber. If the chamber is of excessive depth, the grids are supported about 10 ft below the water surface in order to avoid excessive pressures. One item, sometimes overlooked, is that the carbon dioxide pipe from the compressor should enter the chamber at an elevation above the water surface in order to avoid siphon action. If the inlet and outlet are at about the same elevation, a transverse baffle should be installed to prevent short circuiting of the water.

Various methods of generating carbon dioxide gas are used. At most of the larger plants operated almost continuously, a coke furnace is used to produce carbon dioxide. The heat generated is frequently used for heating water. At smaller plants, especially those not operated continuously, it is difficult to keep a small coke fire burning and some other fuels, such as oil or natural gas, are desirable. Flue gases from coal fires are likely to contain phenols unless the combustion is quite perfect; the designer should be cautious in deciding to use flue gas if the water is to be chlorinated. If coke is used as a fuel the flue gas must be passed through a scrubber and a dryer: if gas is used as a flue, the flue gas need only be passed through a cooling tank. It is very essential that the gas passing to the compressor is dry.

The devices should be sufficient to provide at least 300 lb of carbon dioxide per million gallons of water treated. The capacity of the compressor required varies with the percentage of carbon dioxide in the flue gas, the character of the water treated, and the degree to which recarbonation is to be carried. Careful investigation should be made of the type of compressor to be used.

The recarbonation equipment is one item in a plant for which duplicate units are not absolutely necessary. Since many lime-softening plants have operated for years without recarbonation devices, there is ordinarily no reason why they cannot be discontinued for a few hours or a few days without any unsatisfactory results.

Gravity Filters.—The design of filters for a lime-soda type of water-softening plant is practically the same as the filters of a purification plant. There are several features, however, that require special attention. Filter influent pipes, even where recarbonation is practiced, tend to incrust rapidly with normal carbonates. Such pipes should be designed over-size to permit longer periods between cleaning. They should also be designed with as few changes in

direction as possible; flanged crosses instead of bends should be placed at changes in direction to permit rapid access for cleaning. Where small pipes must be used, they should be provided with flanges or unions so that the entire pipe can be removed for cleaning. Since the influent water is not corrosive, the filter influent pipes may be made of light-weight steel, which is much less expensive for larger sizes than cast-iron pipe. For a similar reason, steel wash-water troughs do not corrode at a lime-softening plant. Since the filter sand will begin to incrust as soon as the plant is in operation, it is obviously unnecessary to specify that the sand shall not be acid soluble as generally called for in the specifications for ordinary filtration plants.

The removal, regrading, and replacement of sand in filters, is an item that is not peculiar to a softening plant but one that has been neglected in the design of both filtration plants and softening plants. A few features included in the design would greatly facilitate handling of filtering material: Ordinarily, it can be conveyed by a water ejector; a high-pressure water line at least 2 in. in diameter should be available at each filter; a permanent pipe line in the pipe gallery should extend to a storage room for filtering materials; there should be provision for temporary rigid or flexible connections to this pipe line at each filter; a room should be provided in which a bin, sufficient in size to hold the filtering material for one filter, can be placed; and, the filtering material from a filter can then be ejected to a sand washer and gravel grader from which the sand and gravel would pass to the storage bins. In returning the filtering material to the filters, the operation is reversed. Provision of such equipment would result in improved operation of plants.

Pressure Filters.—Pressure filters should not be installed unless they cannot be avoided. The sand in such units becomes incrusted, is not frequently inspected, and it is practically impossible to spade it. As a result the water frequently passes through cracks in the filters instead of through the filter media. Since pressure filters are only intended as clarifying devices at lime-softening plants, there is no sanitary objection to their installation where the water to be softened is a well water of satisfactory sanitary quality; pressure filters must not be used where bacterial purification of the water is necessary. The type of plant feasible with pressure filters is frequently of considerably less cost than a plant of similar capacity with gravity filters, and this fact may occasionally justify the use of pressure units.

Zeolite Water Softening

Although zeolite softening units have been used for household and industrial installations for a considerable length of time, the adaptation of this process for municipal water softening is of comparatively recent date. The first municipal plant in Ohio was installed at Lowellville in 1927. Such a plant quite often consists almost entirely of assembled equipment furnished by the manufacturer. For this reason there is frequently very little actual designing to be done by the engineer. Sometimes, his chief duty may be only to determine whether zeolite treatment is the best and most economical process for the water to be treated, and to write specifications so that the plant will be adequate and so that competitive bids will be received.

A number of the earlier municipal zeolite plants were "sold" directly to municipal officials by representatives of equipment companies. In these cases no investigation was made by a disinterested engineer to determine whether the type of plant offered was best suited to local conditions. As a result, some zeolite plants have been installed in places where they are, to say the least, not the most suitable and, in some cases, the plants have been absolutely unsatisfactory. These installations have been, and still are, a detriment to the zeolite-softening field. Reputable manufacturers of zeolite and zeolite equipment now prefer and generally insist that the municipality installing the water-softening plant be represented by a competent engineer experienced in this work.

A zeolite plant has several distinct advantages over a plant of the lime-soda type. The chief merits of this type of plant are: (1) The absence of a sludge disposal problem; (2) lower first cost; (3) simplicity of operation; (4) expert supervision of plant is unnecessary; and (5) single pumping, directly from the well to the distributing system, can be practiced in many instances.

There are also disadvantages to zeolite softening which may be listed as follows: (1) Zeolite softening alone cannot be used where bacterial purification is required; (2) waters containing objectionable proportions of iron and manganese are not suitable for zeolite treatment unless these minerals are removed prior to zeolite softening; (3) some types of zeolite treatment require a large proportion of wash and rinse water; and (4) zeolite treatment alone provides no pH-correction.

Plant Capacity.—It is impossible to rate the daily softening capacity of a zeolite unit by its surface area or by the volume of zeolite in the unit. Its daily capacity depends, among other things, on the number of times the unit is regenerated during 24 hr. Therefore, the engineer must be careful to give proper weight to this factor when considering units of various manufacturers.

Iron Removal Prior to Zeolite Softening.—If the untreated well water has an iron content very much in excess of 0.5 ppm, it is essential to remove it before softening; and it can ordinarily be accomplished by aeration and filtration. Iron and manganese in solution can be removed by base exchange, by regenerating the zeolite with salt. This principle is used at several plants where at least three zeolite units are installed. In these cases, two (or more) units are operated as iron-removal and softening units, while one unit is over-run after its capacity for softening is exhausted. This single unit is thus only effecting iron removal; the mixed water from the three units is delivered to the distributing system in order to avoid serving a water of zero hardness. In this case the operation of the various units is staggered so that each unit in turn is over-run. The writer has observed one plant successfully operated in this manner. At this plant it requires expert operation to deliver a satisfactory water of uniform quality. At some small villages where this type of plant has been installed and where only ordinary operation is provided, the iron content and the hardness of the plant effluent often vary through wide ranges.

pH-*Control.*—Zeolite treatment has no effect whatever on the pH-value. Waters containing iron ordinarily contain large quantities of carbon dioxide. At some plants, where iron removal and water softening are effected by base exchange, the effluent is practically free from iron. However, there has been

no change in the pH-value of the water, and the calcium or incrusting material has been removed; as a result, the water re-dissolves iron either from deposits in the mains or directly from the piping system with the result that the water delivered to the consumer may have a greater iron content than the water delivered directly from the wells.

Adjustment of the pH-value of the water will eliminate these troubles. Aeration of the water will release the carbon dioxide and raise the pH-value. The pH-value of the water can also be adjusted by introducing alkali to neutralize the carbon dioxide. In that case, there must be a constant check on the pH-value of the water delivered. Since the alkali feeders are generally pressure devices similar in principle to the "alum pots" of pressure filters, it is rather difficult to secure a uniform rate of feed. At small plants it is almost impossible to secure the degree of attention necessary to obtain control of the pH-value by alkali feed.

The foregoing operation experiences are cited to emphasize the precautions necessary in the selection of the type of plant to be installed. Zeolite softening has a wide and increasing field. It is important that it shall not be handicapped and discredited because the water is unsuitable or because it has not been properly prepared for zeolite treatment.

Selection of Type of Plant.—After having decided that a zeolite type of water-softening plant is to be installed, the designing engineer must then weigh the merits of the various types. Among the items to be decided are: (*a*) Gravity *versus* pressure units; (*b*) kind of zeolite to be used; and (*c*) up-flow *versus* down-flow units. Some of these questions are inter-related, and a decision on one question may establish the type of unit to be installed. Especially for small plants, it is advisable for the designer to give serious consideration to the installation of the standard pressure units of a reputable manufacturer rather than to install gravity units of his own design.

If pressure units are to be installed, the direction of flow is established by the manufacturer whose equipment is selected. At all except one of the gravity zeolite plants in Ohio the units are the up-flow type. At the one gravity plant, where green sand or natural zeolite is used (synthetic zeolite is used in the others), the units are of the down-flow type.

Gravity zeolite units have (to a lesser degree) the same advantages over pressure units that gravity filters have over pressure filters; namely, that the operation of, and conditions in, the units can be observed at all times. The design of gravity zeolite units has been patterned after the design of gravity, rapid-sand filters. In a down-flow unit the design may be practically identical with the design of a gravity filter. In an up-flow unit the direction of flow is reversed and, therefore, changes in design are necessary. Zeolite softening units are generally deeper than gravity filters. The gravel layers supporting the zeolite are similar to the gravel supporting the sand in a filter. In down-flow units (where green sand is used) the depth of zeolite does not generally exceed 3 ft. In up-flow units the loss of head through them is considerably less than in the down-flow units and the zeolite may be 4 ft to 6 ft deep, and occasionally of even greater depth; similar depths are used for down-flow synthetic zeolite units. Greater depths of zeolite permit economy in the quantity of water

required for washing and rinsing. Frequently, considerations defining the vertical limits of the plant structures, such as the loss of head available and the lowest elevation at which natural drainage can be secured, determine the depth of the zeolite units.

The rate at which water is to be passed through a unit determines the area needed. In down-flow units the rate is generally from 2.5 to 4 gal per sq ft per min. In up-flow units the rate may be 4 to 6 gal per sq ft per min, or even somewhat higher.

In the design of the concrete up-flow softening units there are several significant items. The collecting or effluent troughs (corresponding to the wash-water troughs of a gravity filter) need to be designed only for the maximum rate of flow through the unit. During regeneration of a unit the brine and rinsing water pass downward. The design of the water-distributing system or the rinse water collecting system in the bottom of the unit requires special attention to insure that no brine remains after it is rinsed. A system of perforated laterals such as those used in gravity filters is not satisfactory because the brine is heavy; it would collect below such laterals during regeneration and would gradually pass out after the unit is again placed in service, thus causing a salty water to be delivered from the unit. In order to avoid this condition the strainer system should be designed as a system of laterals with perforations on top of the laterals in which strainers are placed. After the laterals and strainers are installed the entire strainer system should be covered with a layer of grout to an elevation just below the openings. Since the so-called re-wash valve is used every time a unit is regenerated, it should be as accessible as the others.

Proportioning Devices.—It is not satisfactory to deliver the water from the softening units directly to the distributing system since it is of practically zero hardness. Hence, some means must be provided to mix hard water with the effluent from the softening units in order that water of the desired degree of hardness may be delivered to the distributing system.

At a small plant this may be accomplished by installing a hard-water by-pass line on which a valve and a meter are placed. The hardness of the mixed water can be adjusted sufficiently by throttling the valve on the by-pass line. Zeolite equipment companies now furnish automatic proportioning devices to control the quantity of water by-passed; the installation of such devices is almost a necessity for larger plants and is desirable even for small plants.

If the raw water contains objectionable quantities of iron, and if zeolite treatment is provided to remove iron as well as for softening, the by-passed water may also have to be treated for iron removal, in order to prevent the mixed water from having an objectionable iron content. Since the hardness of a well water varies only slightly throughout the year, the proportioning devices need only to be changed infrequently after they are once set.

Salt and Brine Devices.—A considerable economy can be effected if salt is purchased in bulk instead of in bags. For this reason almost all plant administrators purchase salt in bulk. A desirable method of storing salt is to provide a basin into which it can be dumped from a truck or a railroad car. Such a basin should be provided with a tight cover, preferably of concrete. It

should have a capacity sufficient to hold at least a month's supply, and should be kept filled with water. The salt rests on layers of sand and gravel over the floor which should drain to a gutter in which there are half tile or other brine-collecting devices.

At most plants, where there are gravity zeolite units, the concentrated brine passes from the salt-storage tank to a brine-measuring tank where the brine is diluted with water to the desired strength for regenerating the zeolite. The measuring tank is usually of a capacity sufficient to provide for the regeneration of one unit. If the brine cannot pass by gravity from the salt-storage tank to the brine-measuring tank, it is desirable to place the brine pump at such an elevation that there will always be a positive head on the suction of the pump. The concentrated brine may be ejected to a softening unit; in this case, the water from the ejector provides the dilution to reduce the brine to the desired strength for regeneration.

Back-Wash and Rinsing.—Down-flow units must be back-washed in the same manner as gravity filters, although generally at a lesser rate. If the hard water is clear and free from iron, it may be desirable to back-wash and to rinse the units with hard water instead of softened water. The designing engineer should secure accurate information regarding the percentage of water required for back-washing and rinsing. This feature is a factor that is often neglected in deciding the relative merits of lime-soda *versus* zeolite softening.

Conclusions

The public in Ohio is just becoming aware of the fact that softened water can be made available in almost any locality at a cost only slightly greater than the rates being charged for hard water. The field for water softening is practically unlimited. This offers water-works engineers an opportunity to serve the public; and it also offers an opportunity to design and install treatment plants for municipalities where previously it has been thought that water-supply and water-treatment problems had been definitely solved.

Since it is desirable to profit as much as possible by past experiences, the writer offers the comments in his paper in the hope that they may be of assistance to engineers engaged in this type of work. In accordance with Society policy, he has purposely refrained from comments on specific patented methods, processes, or equipment. It is suggested that discussers observe the same limitations.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

AERODYNAMICS OF THE PERISPHERE AND TRYLON AT WORLD'S FAIR

By Alexander Klemin,[1] Esq., Everett B. Schaefer,[2] Esq., and J. G. Beerer, Jr.,[3] Esq.

Synopsis

There can be no better way of beginning this paper than to quote the following recommendation from the pamphlet, entitled "Standards of Design for Structural Steel," (1)[4] published by the United States Navy Department:

"For structures of unusual character or magnitude where the wind is an important factor, the actual distribution and intensity of positive and negative wind pressure should be investigated, preferably by wind tunnel tests of a scale model, and the stresses calculated on that basis should be used if they exceed those determined from the formula given herein."

Certainly the Perisphere and the Trylon, the "theme" buildings at the New York World's Fair, come under the designation of structures of unusual character and magnitude—since the Perisphere is a hollow spherical building with a diameter of 200 ft, and the Trylon is a tapered triangular prism-like object, 675 ft high. The group, with a variety of supports for the sphere collars and columns, is shown in Fig. 1. The difficulty in estimating the wind loads, and especially their distribution, on these two buildings was all the greater since they are in close proximity to one another. Because of mutual interference they must be considered as a single aerodynamic unit, since no prior experimentation was available as to the aerodynamic effects on a sphere close to the ground, and since, in general, the contemplated project is without parallel in structural history.

Accordingly, the senior writer immediately suggested the advisability of a systematic wind-tunnel investigation. The paper is an account of these experiments, accompanied by theoretical considerations.

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by **September 15, 1938.**

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[4] Numerals in parentheses, thus (1), refer to items in the Appendix, List of References.

The experimental results constitute an addition to the meager information available on the aerodynamics of buildings, and may find application in the design of other structures, such as circular gas tanks, triangular towers, etc. In general, the technique of the investigation should be of interest to structural designers who have to deal with wind loads.

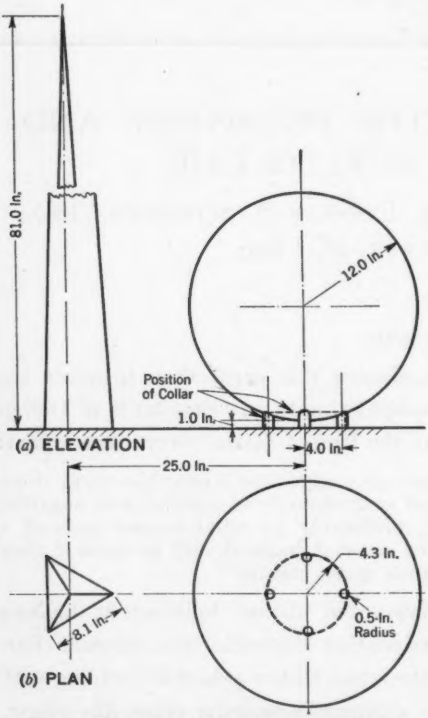


FIG. 1.—MODELS OF PERISPHERE AND TRYLON (MODEL SCALE, 1 : 100)

From a theoretical point of view the main interest of the paper lies in the mathematical study of the air flow around a sphere when in proximity with the ground, and the comparison of the calculated results with those obtained experimentally.

THEORETICAL CONSIDERATIONS

In examining the results of the tests of pressure distribution over the Perisphere, a useful check is provided by theoretical calculations of the potential flow past a sphere in free air, and of the potential flow past a sphere close to the ground. There is also scientific interest in comparing the actual flow with the potential flow.

Flow Past a Sphere in Free Air.—For a sphere moving in free air with Velocity V, the velocity potential is given by,

$$\phi = -\frac{V\,a^3 \cos\theta}{2\,r^2} \qquad (1)$$

in which, a = radius of the sphere; r = distance from the origin to any point; and θ = angle between Vector r (Fig. 2) and the direction of motion of the sphere. If the sphere is at rest, and the air is flowing past it with Velocity V, the velocity potential becomes,

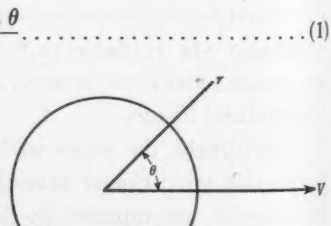


FIG. 2

$$\phi = -\frac{V\,a^3 \cos\theta}{2\,r^2} - V\,r\cos\theta \qquad (2)$$

The velocity at right angles to Vector r is, therefore,

$$V_n = \frac{\partial \phi}{r \, \partial \theta} = \left[\frac{V a^3}{2 r^3} + V r \right] \sin \theta \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and over the surface of the sphere, the tangential velocity over any great circle whose diameter is along the direction of the general flow is,

$$V_t = \frac{3}{2} V \sin \theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

with a maximum velocity of $\frac{3}{2} V$; in which, $\theta = 90$ degrees.

If p_∞ is the static pressure at infinity, where the flow is undisturbed by the presence of the sphere, and p_n is the normal pressure at any point on the sphere, then by Bernoulli's equation,

$$p_\infty + \rho \frac{V^2}{2} = p_n + \frac{V_n^2}{2} \ldots (5)$$

and,

$$p_n - p_\infty = \rho \frac{V^2}{2} - \rho \frac{V_n^2}{2}$$

$$= \rho \frac{V^2}{2} \left[1 - \left(\frac{3}{2} \sin \theta \right)^2 \right] \ldots (6)$$

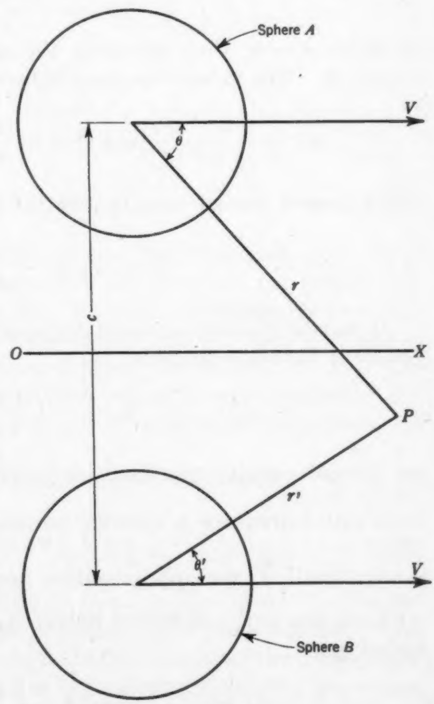


FIG. 3

Two Spheres of Equal Diameter Moving with Equal Velocity Along Parallel Lines.—In this case (see Fig. 3) it is clear that there will be no flow across the horizontal plane whose trace is the axis, $O\,X$. Therefore, the flow will be the same as if Sphere A or Sphere B were moving with the same velocity, V, in the presence of the boundary or ground, $O\,X$. To find the velocity potential in this case is the best introduction, therefore, to the problem of the flow past a sphere in the presence of the ground. In finding this velocity potential, an approximate treatment must be adopted because of the mathematical difficulties involved.

Thus, if the velocity potential, ϕ_1, for Sphere A, moving with Velocity V, is expressed by Equation (1), then near Sphere B (see Fig. 2):

$$\phi_1 = - \frac{V}{2} \frac{a^3 \, r \cos \theta}{r^3} = - \frac{V}{2} a^3 \frac{r' \cos \theta'}{r^3} = \frac{V}{2} \frac{a^3 \, r' \cos \theta'}{c^3} \ldots\ldots\ldots (7)$$

since $r \cos \theta = r' \cos \theta'$. The normal velocity over Sphere B is,

$$V_n = \frac{\partial \phi'}{\partial r'} = -\frac{V}{2}\frac{a^3 \cos \theta'}{c^3} \quad \text{.................... (8)}$$

that is, into Sphere B. Since there can be no velocity normal to a solid body, a space doublet is applied at the center of Sphere B having a velocity potential,

$$-\left[\frac{V}{2}\frac{a^3}{c^3}\right]\frac{a^3}{2(r')^2}\cos\theta' = \phi_2 \quad \text{.................. (9)}$$

which induces a velocity,

$$\frac{-\partial\left(\frac{V}{2}\frac{a^3}{c^3}\frac{a^3}{2\,(r')^2}\cos\theta'\right)}{\partial r} = \frac{V}{2}\frac{a^3\cos\theta'}{c^3} \quad \text{.............. (10)}$$

in which $r = a$; thus canceling the normal velocity across the surface of Sphere B. The value of ϕ_2 near Sphere A is (approximately):

$$\phi_2 = -\frac{V}{4}\frac{a^6}{c^3\,(r')^2}\cos\theta' = -\frac{V}{4}\frac{a^6\,r'\cos\theta'}{c^3(r')^3} = -\frac{V}{4}\frac{a^6\,r\cos\theta}{c^6} \quad \text{.... (11)}$$

which gives a normal velocity over the surface of Sphere A equal to,

$$V_n = -\frac{V}{4}\frac{a^6\cos\theta}{c^6} \quad \text{...................... (12)}$$

It can be readily seen that by applying a space doublet at the center of Sphere A having a potential,

$$\phi_3 = -\left(\frac{V}{4}\frac{a^6}{c^6}\right)\frac{a^3}{r^2}\cos\theta \quad \text{.................... (13)}$$

the normal velocity expressed by Equation (12) will be canceled. Although this again introduces a velocity normal to Sphere B $\left(\text{since } c > a\text{; and } \frac{a^6}{c^6}\right.$ is very small $\left.\right)$, the approximation need be carried no further, and one can say that, due to the motion of Sphere A, the velocity potential near it is represented by,

$$\phi_4 = -\frac{V}{2\,r^2}a^3\cos\theta - \frac{V}{4}\frac{a^6\,r\cos\theta}{c^6} - \frac{V}{8}\frac{a^6}{c^6}\frac{a^3}{r^2}\cos\theta \quad \text{........ (14)}$$

and also that, due to the motion of Sphere A, the velocity potential near Sphere B may be represented with sufficient approximation by,

$$\phi_5 = -\frac{V}{2}\frac{a^3\,r'\cos\theta'}{c^3} - \frac{V}{4}\frac{a^6}{c^3\,2\,(r')^2}\cos\theta' \quad \text{............ (15)}$$

Hence, the velocity potential near Sphere A due to the combined motion of

both spheres must be, by considerations of symmetry,

$$\phi_6 = -\frac{V}{2\,r^2}\,a^3\cos\theta - \frac{V}{4}\,\frac{a^6\,r\cos\theta}{c^6} - \frac{V}{8}\,\frac{a^6\,a^3}{c^6\,r^2}\cos\theta - \frac{V}{2}\,\frac{a^3\,r\cos\theta}{c^3}$$
$$- \frac{V}{4}\,\frac{a^6}{c^3}\,\frac{\cos\theta}{r^2} \qquad (16)$$

Air Flow Past a Sphere in Presence of the Ground.—To bring the air into motion and the spheres to rest, it is now only necessary to add the term $-V\,r\cos\theta$ to Equation (16) and the complete expression for ϕ_7 is the result. When $r = a$,

$$\phi_7 = -\frac{V}{2}\,a\cos\theta - \frac{V}{4}\,\frac{a^6}{c^6}\,a\cos\theta - \frac{V}{8}\,\frac{a^7}{c^6}\cos\theta - \frac{V}{2}\,\frac{a^4\cos\theta}{c^3}$$
$$- \frac{V}{4}\,\frac{a^4}{c^3}\cos\theta - V\,a\cos\theta \qquad (17)$$

and the tangential velocity is:

$$\frac{\partial\phi}{a\,\partial\theta} = V\sin\theta\left[\frac{1}{2} + \frac{1}{4}\,\frac{a^6}{c^6} + \frac{1}{8}\,\frac{a^6}{c^6} + \frac{1}{2}\,\frac{a^3}{c^3} + \frac{1}{4}\,\frac{a^3}{c^3} + 1\right]$$
$$= \frac{3}{2}\,V\sin\theta\left[1 + \frac{1}{2}\,\frac{a^3}{c^3} + \frac{1}{4}\,\frac{a^6}{c^6}\right] \qquad (18)$$

For maximum velocity $\theta = 90°$ in Equation (18), and $\sin\theta = 1$. It is seen that the tangential velocity is greater for the sphere in the presence of the ground, than for the sphere in free air. Another interesting phenomenon is that the velocity, theoretically, is dependent only on the value of θ, and is the same at symmetrical points above and below the sphere, when in the presence of the ground, and that the stagnation points are not changed by the ground board. Furthermore, the velocity will be greater as the distance, c, between the centers becomes smaller.

The pressure at any point can be determined from the formula,

$$p_n - p_\infty = \rho\,\frac{V^2}{2}\left\{1 - \left[\frac{3}{2}\sin\theta\left(1 + \frac{1}{2}\,\frac{a^3}{c^3} + \frac{1}{4}\,\frac{a^6}{c^6}\right)\right]^2\right\} \qquad (19)$$

Points at Which the Tangential Velocity Is Equal to That in the Undisturbed Air Stream.—In the examination of test results, it is of some interest to determine where the tangential velocity is equal to the velocity of the undisturbed air stream. This occurs when $p_n - p_\infty$, as expressed by Equation (6), equals zero; that is, when $\theta = 41.7$ degrees. For the sphere in proximity with the ground, substituting $[1 + K]$ for $\left(1 + \frac{1}{2}\frac{a^3}{c^3} + \frac{1}{4}\frac{a^6}{c^6}\right)$ in Equation (19), the value of θ will be given by,

$$1 - \left[\frac{3}{2}\sin\theta\,(1 + K)\right]^2 = 0 \qquad (20)$$

so that θ will have a smaller value than for the sphere alone.

EXPERIMENTAL METHODS

Model Dimensions and Reynolds Number of the Tests.—The wind-tunnel models of the Perisphere and Trylon were built to a scale of 1 : 100, with only external appearance represented. The main model dimensions were: Sphere diameter, 2.00 ft; height above ground board, 0.08 ft; Trylon height, 6.75 ft; and, Trylon width at the base, 0.68 ft.

In aerodynamic experimentation, scale effects (that is, the difference between air flow over the model and the air flow over the larger full-scale object) must always be considered. The effective scale of the model is represented by the Reynolds number, R, which in the case of the sphere assumes the form,

$$\mathsf{R} = \frac{V\,D}{\nu} \qquad (21)$$

in which V = velocity of the air stream, in feet per second; D = diameter of the sphere; and, ν = coefficient of kinematic viscosity.

The resistance coefficient of a sphere is conveniently represented by the equation,

$$C_D = \frac{R}{\left(\frac{\rho\,V^2}{2}\right)\frac{\pi\,D^2}{4}} \qquad (22)$$

in which R = resistance of the sphere, in pounds; ρ = density of the air $= \frac{\text{specific weight per cubic foot}}{g}$; and, D = diameter of the sphere, in feet.

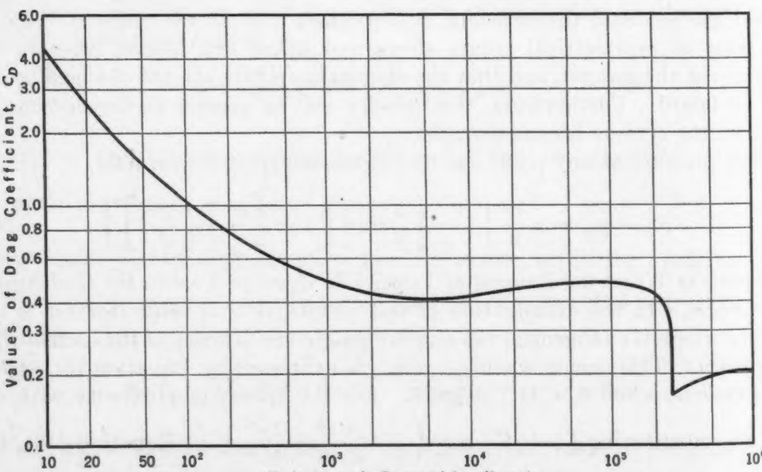


FIG. 4.—VARIATION IN SPHERE DRAG COEFFICIENT WITH REYNOLDS NUMBER

In Fig. 4 the values of the drag coefficients are plotted against the Reynolds number, the curve being obtained from prior tests. It will be seen that, although there is considerable variation in C_D at the lower Reynolds number,

the tests at the higher values of **R** are definitely in an asymptotic region. This indicates that the critical region in which the character of the flow changes has been passed, and that the pressure distribution tests, made at an effective Reynolds number of 1 833 880, were definitely in the region where the flow has assumed the character of complete turbulence in the boundary layer. This indicates that the results of the pressure distribution tests may be applied with confidence to full-scale conditions.

It was only possible to make the force tests on the sphere up to a maximum of 30 miles per hr, but in the pressure distribution tests, where it was possible to brace more securely, the indicated air speed of the tunnel was 70 miles per hr, giving a Reynolds number, $\frac{V\,D}{\nu}$, equal to $6\,378\,(70 \times 1.467) \times 2 = 1\,390\,914$.

The "Effective Reynolds Number" is the best basis of comparison between free air and wind-tunnel results since it includes the effect of the tunnel turbulence, and is defined as the actual Reynolds number multiplied by the turbulence factor which, in this case, equals $1\,390\,914 \times 1.4 = 1\,833\,880$ since the turbulence factor of the 9-ft wind tunnel was 1.4.

Because of the large forces encountered, the drag measurements of the needle were made at lower air speeds than the pressure distribution tests on the sphere, namely 40, 50, and 59.4 miles per hr. Since the needle is not a stream-lined body, its resistance does not change appreciably with the Reynolds number and, therefore, the results of the tests are also applicable to full scale without correction.

Methods of Measuring Air Forces.—The methods of measuring air forces were entirely conventional and are illustrated by Fig. 5. Fig. 5(*a*) shows the sphere suspended in the presence of the "ground board" which was used to simulate the ground effect experienced by the Perisphere. Force tests for the

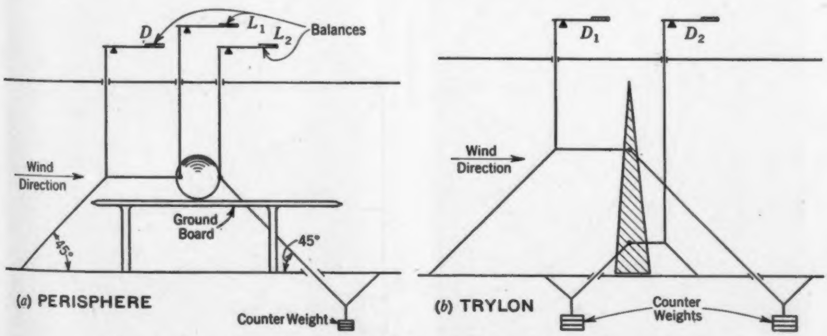


Fig. 5.—Schematic Diagram of Suspension System for Force Tests

sphere were also made on the same set-up with the ground board removed, to simulate "free air" conditions. With the arrangement shown in Fig. 5(*a*), the sum of the readings of Balances L_1 and L_2 gives the vertical or lift force on the sphere. A sufficiently heavy counterweight is used as shown, so that the wires leading to the lift balances are always in tension. The front wire takes

up all the drag, and since it is inclined at 45° to the horizontal, the vertical wire running to the front balance, *D*, transmits a force equal to the drag or air resistance. It is clear also that with this arrangement moments can be computed readily about any point. "Tare" measurements and corrections are made without the wind stream, in obvious fashion.

Drag tests of the Trylon, or needle, were run with the needle in the presence, but just free, of the ground, with the tunnel floor representing the ground The special mounting of the Trylon during the drag tests is illustrated diagrammatically in Fig. 5(*b*).

The vertical center of pressure (point of application of resultant drag force) is determined by obvious calculation when the forces in the two drag wires and their points of application are known.

Measurement of Pressure Distribution on Sphere, (*2*), (*3*).—The pressure distribution was measured on one hemisphere only as it may be assumed that the air flow is symmetrical about a plane through the center of the sphere, normal to the ground line and parallel to the direction of the air stream. This system was particularly convenient inasmuch as it permitted one-half the sphere to be free of pressure orifices, thus leaving a clear space for the exit of the pressure tubes, and avoiding flow disturbing influences.

The pressures on the surface of the hemisphere were measured at thirty-seven points disposed as follows (see Fig. 6): All pressure holes lay on the

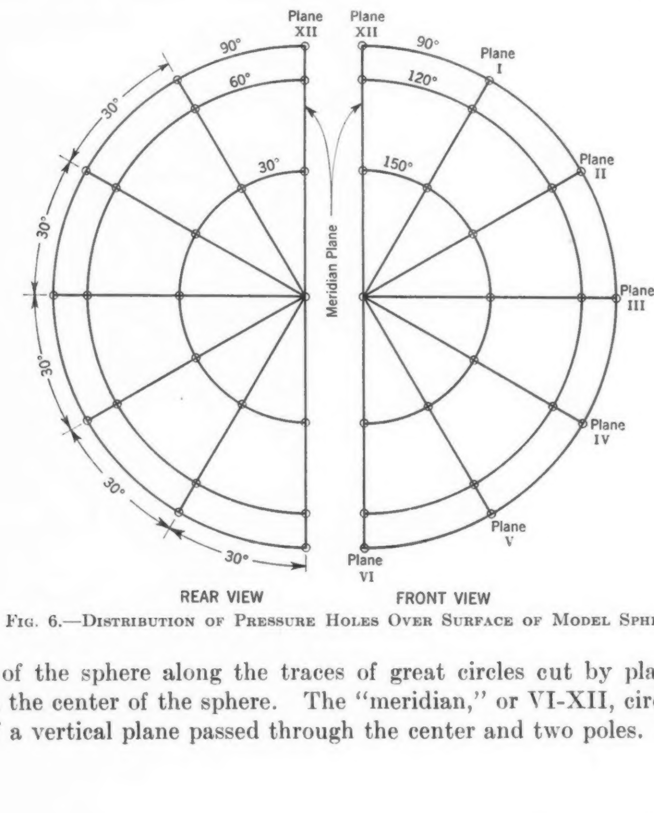


FIG. 6.—DISTRIBUTION OF PRESSURE HOLES OVER SURFACE OF MODEL SPHERE

surface of the sphere along the traces of great circles cut by planes passed through the center of the sphere. The "meridian," or VI-XII, circle was the trace of a vertical plane passed through the center and two poles. When the

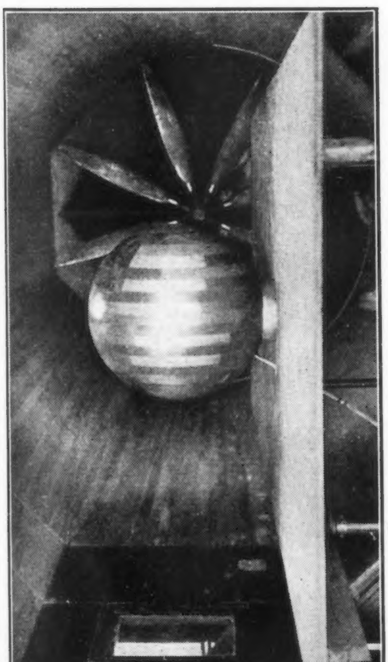


FIG. 7.—GENERAL ARRANGEMENT OF MODEL IN TUNNEL

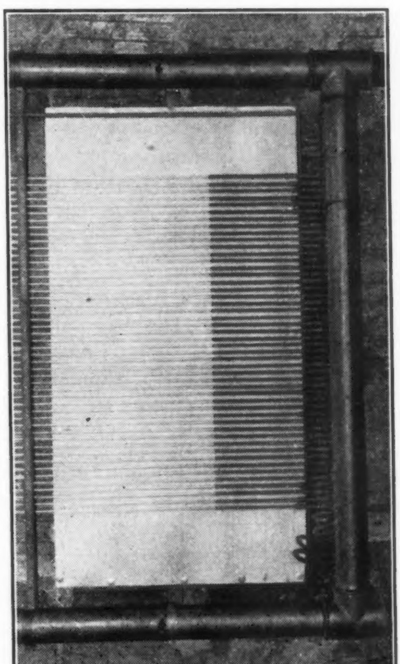


FIG. 8.—MULTIPLE MANOMETER FOR PRESSURE MEASUREMENT

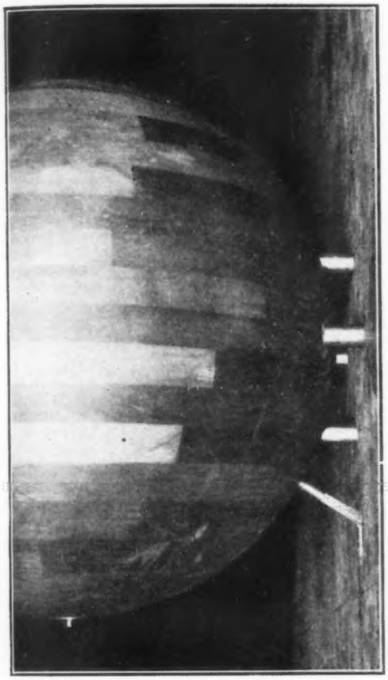


FIG. 9.—SPHERE ON COLUMNS

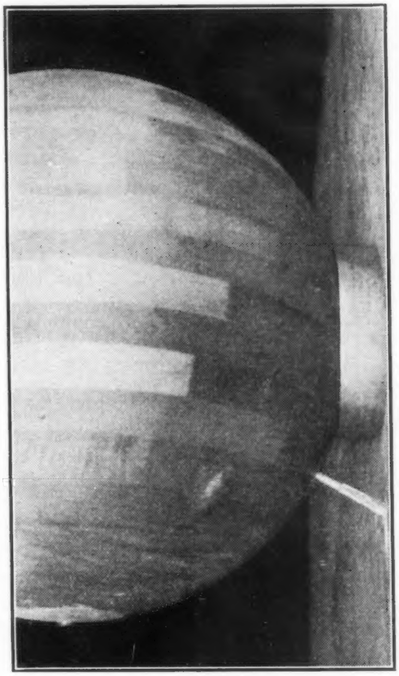


FIG. 10.—SPHERE ON COLLAR

sphere was mounted in the tunnel this "meridian" plane was coincident with the vertical plane of symmetry of the tunnel. The planes designated as Planes I, II, III, IV, and V were all at 30° intervals from the "meridian" plane. The pressure holes on each great circle were placed at 30° intervals, with two points common to the "meridian," or VI-XII, plane and to all other planes. This system of defining planes and points was selected because theoretical calculations are made conveniently for points thus arranged on the surface of a sphere.

The pressure orifices were small brass tubes, set flush with the sphere's surface, gathered together in the hollow interior, and led out through a single hole on the far side of the surface that did not contain pressure holes.

The leads from the pressure orifices were connected to the multiple-tube manometer with a common fluid reservoir. There were several additional pressure leads connected into the manometer which were used to measure the static pressure of the undisturbed air stream in the test chamber in order to have a basic reference for the calculation of the absolute pressures on the sphere surface. The pressures of the undisturbed air stream were measured on the sides of the channel well ahead of the position of the sphere. A permanent record of the fluid-pressure heads indicated by the colored liquid in the glass manometer tubes was obtained by means of positive printing paper placed behind the tubes and exposed to a carbon arc light. The use of this procedure made it quite a simple matter to run a series of pressure tests, take the permanent records of each, and, later, make the conversions to the desired pressure units.

Fig. 7 shows the general arrangement of the model in the wind tunnel; Fig. 8, the multiple manometer set-up outside the tunnel; Fig. 9, the model on the four-column supports; and Fig. 10, the model on the collar support. Pressure distribution tests were made with the sphere suspended without the ground board (to simulate what may be called "free air conditions"). Subsequently, the sphere was tested in the presence of the ground board; in other runs, it was tested for pressure distribution in the presence of the ground board and the Trylon combined.

Force Tests on Trylon

Drag Tests on the Trylon.—The tests on the Trylon were made in two attitudes relative to the wind direction (see Fig. 11): (1) With one of the pointed edges into the wind; and (2) with one of the flat sides into the wind. It was realized, of course, that the greatest air resistance would occur with a flat side into the wind, but the condition of a pointed edge into the wind was deemed worthy of investigation. The results of the drag tests on the Trylon may best be expressed in the form of coefficients of drag, as exemplified by the American absolute drag coefficient, C_D, of aeronautical practice, which is defined similarly to Equation (22) for the sphere, namely,

$$C_D = \frac{D}{\left(\rho \frac{V^2}{2}\right) S} \qquad (23)$$

in which S = the area of a flat side projected on a vertical plane, in square feet. Although the drag tests of the Trylon were run at several tunnel air speeds, the coefficients for all speeds were sufficiently close to permit an average value to be chosen for each of the two attitudes tested. This coincidence of values had been expected on theoretical grounds. The values are:

Attitude of Trylon	Drag Coefficient, C_D
Pointed edge into wind	0.879
Flat side into wind	1.433

They substantiated the anticipation that the Trylon with the flat side into the wind would show the greater drag, since the body with an edge into the wind is really of crudely semi-streamlined form, whereas with the flat side into the wind the flat-plate effect is predominant. The accepted coefficient for the flat-plate drag is $C_D = 1.28$, as used in aeronautical engineering practice. The value obtained for the Trylon with the flat side into the wind is $C_D = 1.433$, about 12% greater than for the flat plate alone. The difference between the two may be justified in view of the fact that the Trylon, being a solid of three dimensions, presents some surface area to the wind stream, so that, in addition to the flat-plate drag effect, there is a drag due to the surface areas of the body, with the probability of turbulent flow on the two down-stream sides.

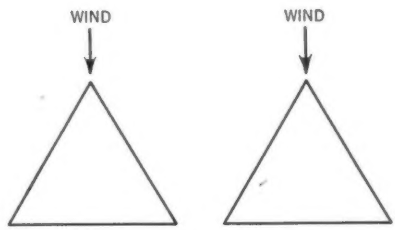


FIG. 11.—TWO POSITIONS OF TRYLON RELATIVE TO WIND

The center of pressure position of the drag forces acting on the Trylon, with a flat side into the wind, was shown to be at an average value of 28.50 in. from the base and along the vertical axis. This point too seems logical since it is within 1.5 in. of the centroid of area of the vertically projected triangle of a flat side of the Trylon.

The full-scale air resistance or drag and upsetting moment about the base of the Trylon may be computed easily for any given air speed by substitution in Equation (23) for the solution of force, and the moment by the usual formula, in which the center of pressure position shown by the tests would be used as the moment arm. If it is assumed that the air speed is 100 miles per hr, the value, $C_D = 825\,000$ lb, would be obtained for the full-scale body with flat side into the wind, and an area projected on the vertical of 22 500 sq ft.

The wind load at 100 miles per hr is thus seen to be 36.6 lb per sq ft of projected area. For buildings higher than 100 ft the Building Code ordinances of New York, N. Y., call for an assumed wind pressure of 20 lb per sq ft of exposed surface from the top of the building down to the 100-ft level. If the flat-plate coefficient of 1.28 is used, this is equivalent to 78 miles per hr. The results of the tests on the Trylon, computed for a wind of 78 miles per hr, would show a loading of about 22.3 lb per sq ft for the entire length of the projected area.

Force Tests on the Perisphere

Drag Tests in Free Air.—The coefficients of drag, C_D, for the free air test (that is, without the presence of the ground) are given in Table 1(*a*). These

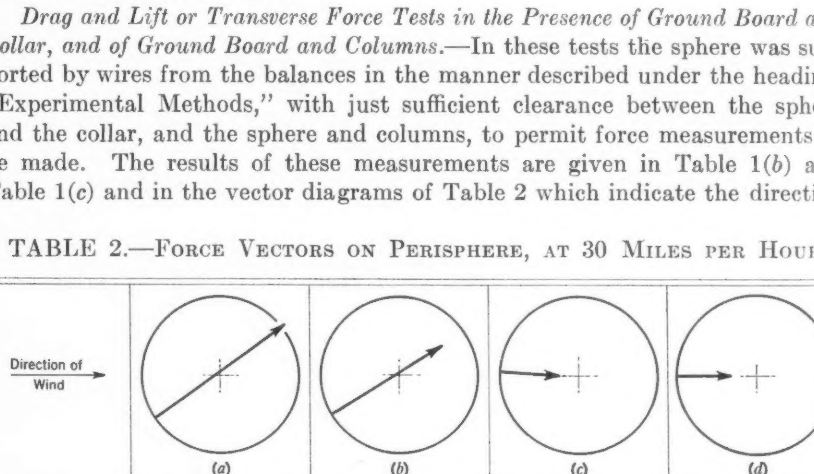
TABLE 1.—Force Tests on Perisphere

Air speed, in miles per hour	(*a*) In Free Air		(*b*) With Collar and Ground Board			(*c*) With Columns and Ground Board		
	Drag, *D*, in pounds	Coefficient, C_D	Drag, *D*, in pounds	Lift, *L*, in pounds	Coefficient, C_D	Drag, *D*, in pounds	Lift, *L*, in pounds	Coefficient, C_D
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
15	0.305	0.169	1.040	0.73	0.573	0.885	0.45	0.489
20	0.620	0.193	1.875	1.34	0.583	1.560	0.92	0.485
25	0.930	0.185	2.880	2.07	0.573	2.455	1.47	0.488
30	1.445	0.200	4.140	2.98	0.573	3.500	2.14	0.485

values agree with results obtained in other laboratories, and they indicate the attainment of an asymptotic condition of flow which is not greatly altered by transposition to full scale. No transverse or lift force is found, of course, under this condition of test.

Drag and Lift or Transverse Force Tests in the Presence of Ground Board and Collar, and of Ground Board and Columns.—In these tests the sphere was supported by wires from the balances in the manner described under the heading, "Experimental Methods," with just sufficient clearance between the sphere and the collar, and the sphere and columns, to permit force measurements to be made. The results of these measurements are given in Table 1(*b*) and Table 1(*c*) and in the vector diagrams of Table 2 which indicate the direction

TABLE 2.—Force Vectors on Perisphere, at 30 Miles per Hour



Item No.	Description	In the Presence of: Ground board and collar (*a*)	Ground board and columns (*b*)	Ground board only (*c*)	Free air test (*d*)
1	Coefficient of drag, C_D	0.573	0.485	0.300	0.200
2	Resultant force vector, C_R	0.705	0.569	0.303	0.200
3	Angle θ between C_R and the horizontal plane	35°.8	31°.4	−5°.77	0°
4	Perpendicular distance, *e*, from the center of the sphere to C_R, in inches	0.30	0.27	0.24	0

of the resultant force. This resultant force is expressed in coefficient form as C_R, the coefficient being defined by an equation similar to that used for C_D (see Equation (23)), but with the resultant force, R, used instead of the drag, D. The resultant force has been calculated in the usual manner from the drag, D, and the transverse or lift force, L.

It is clear that the introduction of the ground board and the collar, or of the ground board and columns, serves to produce an appreciable transverse force in an upward direction—that is, a "lift" in aeronautical terminology. The production of such transverse forces is readily explainable. The introduction of the collar below the sphere naturally retards the flow on the lower surface, and also removes suction effects over a considerable part of the lower area. The presence of the column produces a similar effect but not to so large an extent, and hence the resultant vector is less inclined away from the horizontal in the case of the columns, than in the case of the collar, which blocks the air flow so effectively. It will also be noticed that for the sphere in the presence of the ground only (that is, without collar or columns (Table 2(*c*)), the transverse force is small, but in a downward direction as shown by the resultant force vector which points downward. It is apparent that for this case the air flow is speeded up between the ground board and the lower half of the sphere giving rise to high suction effects in this region and, hence, a downward or negative lift force.

The foregoing conclusions seem to be substantiated by the pressure distribution tests for all the cases, the results of which are given subsequently under the heading, "Pressure Distribution Tests on Sphere." The transverse forces are probably of little importance to the structural engineer. It is most interesting, however, to compare the drag coefficients under the various conditions of the test (see Item No. 1, Table 2).

The great increase in the drag is explainable partly by the increase in speed over the surface of the sphere when in the presence of the ground (as indicated under "Theoretical Considerations" and substantiated by the pressure distribution tests), partly by the general disturbance of the flow due to the obstructions, and partly by the additional or induced drag which follows on the production of lift.

From a structural engineer's point of view it is important to note that wind forces on the sphere are greater in the case of the collar than in the case of the columns, although even in the very worst condition the direct drag force on the sphere is less than one-half the force that would be experienced on a flat plate of the same area as the projected area of the sphere.

Pressure Distribution Tests on Sphere

The following pressure distribution tests were run:

Test No.	Description
1	Free air test of sphere alone.
2	Sphere alone in the presence of the ground board.
3	Sphere on four columns, 1 in. in diameter, with ground board.

Test No.	Description
4	Sphere on collar, 8 in. in diameter, with ground board.
5	Sphere on collar, with ground board and with the Trylon 25 in. on one side.
6	Sphere on collar, with ground board and with the Trylon 25 in. in front.

The distance of 25 in. was measured from the center of the base of the Trylon to the center line of the sphere as shown in Fig. 1. The results of these pressure distribution tests are best shown by the diagrams in Figs. 12, 13, and 14. The pressure distribution diagrams for all the tests are plotted around 360° for the meridian plane (VI-XII) and around 360° for the equator plane (III-IX). In addition, corresponding diagrams have been plotted for Planes I-VII and II-VIII for the test of the sphere alone in the presence of the ground board, so that the pressure distribution over the entire sphere is represented for that case as shown in Figs. 13(*a*), 13(*b*), 13(*c*), and 13(*d*).

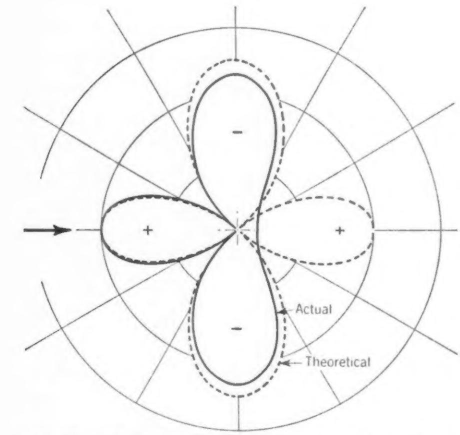


FIG. 12.—PRESSURE DISTRIBUTION OVER ANY GREAT CIRCLE PLANE OF SPHERE IN FREE AIR

The method of plotting the results of the pressure distribution tests utilizes a form of polar diagram in which the wind direction is indicated by the heavy arrow, the radial lines, the 30° divisions, on which the pressures were measured, and the concentric circles, represent radii of $\left(\frac{p_n - p_\infty}{\rho \frac{V^2}{2}}\right)$ equal to 0.5, 1.0, and 1.5, respectively. The expression, $\frac{p_n - p_\infty}{\rho \frac{V^2}{2}}$, is the same as that used in Equation (19). The test data were reduced to this pressure-unit ratio because it is non-dimensional and, therefore, will be the same for all velocities beyond the critical range. It will be noted that on the pressure plots certain parts of the distribution curves are marked plus (+) and others minus (−). The plus (+) sign means that the pressure-unit ratio was positive, thereby indicating a region of positive pressure acting normal to the sphere surface; and, the minus (−) sign shows the negative pressure or suction area normal to the sphere surface. Pressures, of course, are referred to the static pressure of the undisturbed air stream as a base.

Under the heading, "Theoretical Considerations," the flow for a perfect fluid was investigated for the cases of the free-air sphere, and for the sphere in

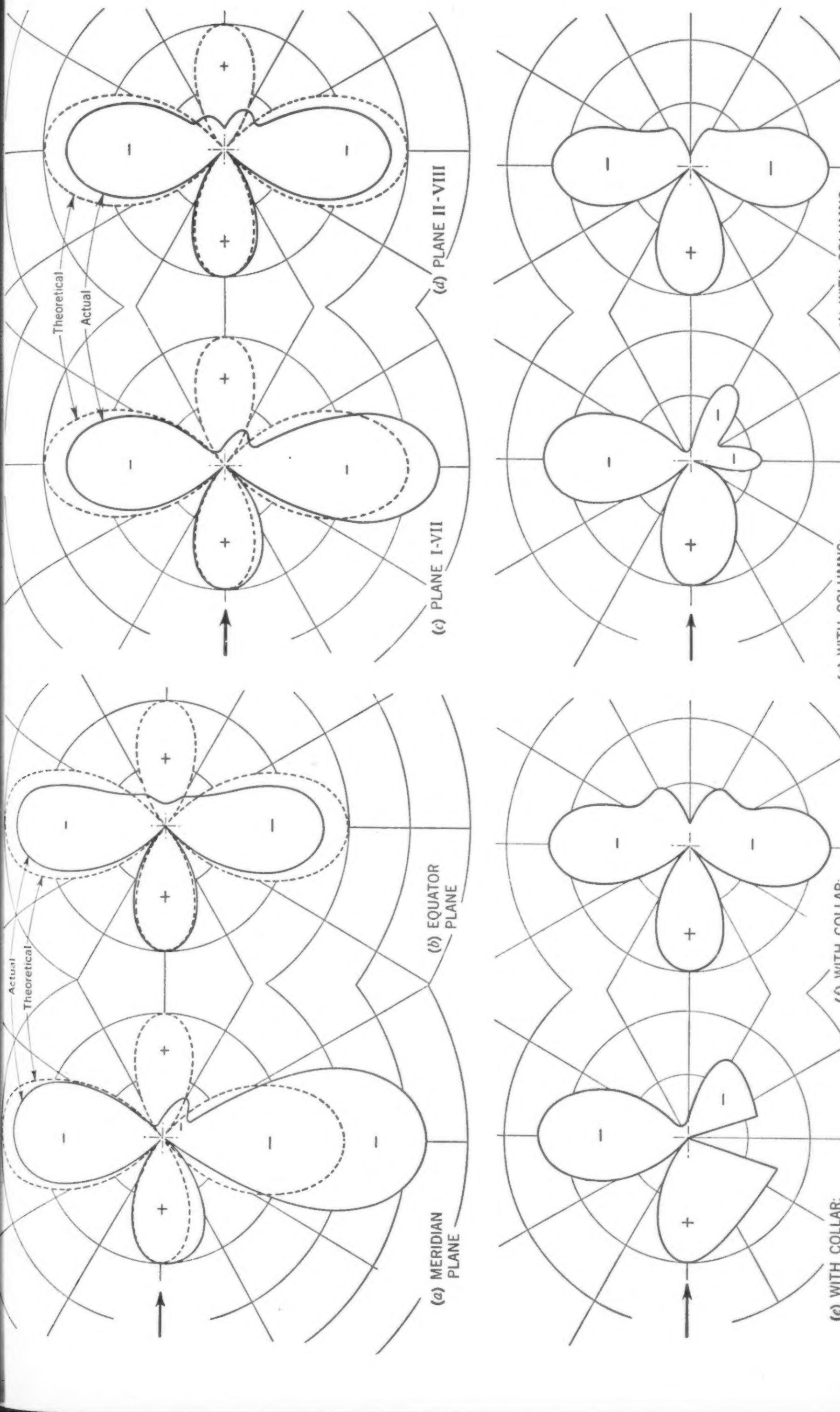


FIG. 13.—PRESSURE DISTRIBUTION OVER THE SPHERE; IN THE PRESENCE OF THE GROUND BOARD

the presence of the ground board. The pressure ratios have been computed by Equations (6) and (18), and it is interesting at this point to compare the theoretically calculated and the measured pressures over the sphere. Therefore, in Figs. 12, 13(*a*), 13(*b*), 13(*c*), and 13(*d*) the measured pressures are shown by the full line and the theoretical pressures by the dotted line.

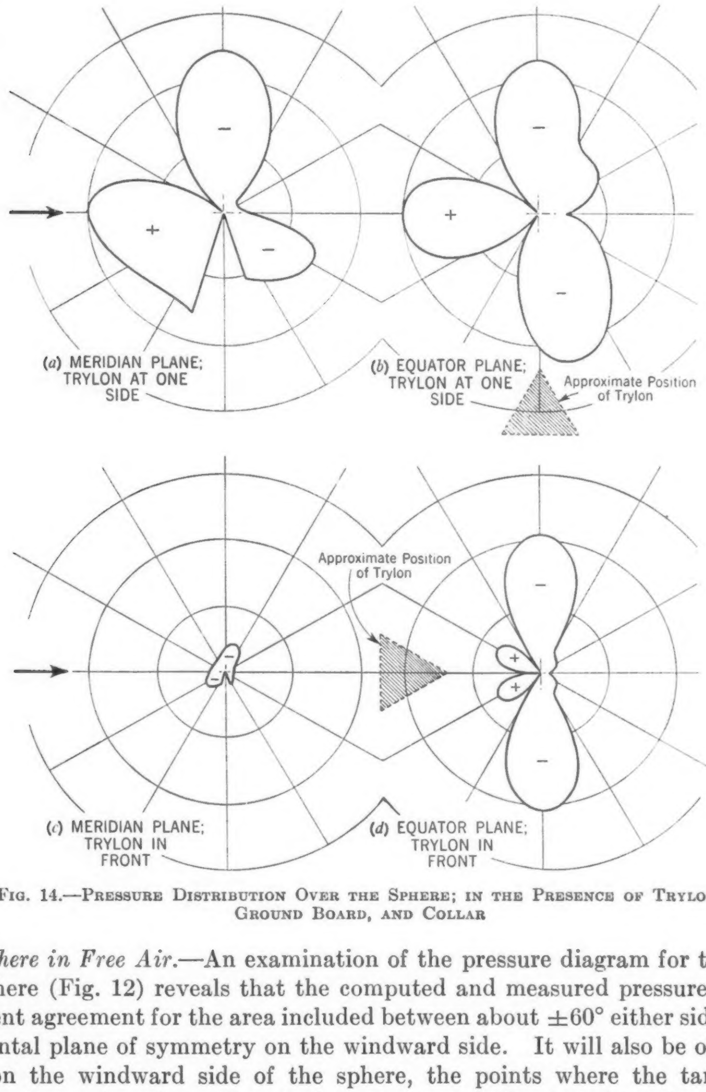


FIG. 14.—PRESSURE DISTRIBUTION OVER THE SPHERE; IN THE PRESENCE OF TRYLON, GROUND BOARD, AND COLLAR

Sphere in Free Air.—An examination of the pressure diagram for the free-air sphere (Fig. 12) reveals that the computed and measured pressures are in excellent agreement for the area included between about $\pm 60°$ either side of the horizontal plane of symmetry on the windward side. It will also be observed that on the windward side of the sphere, the points where the tangential velocity is equal to the undisturbed air stream velocity (or where the pressure ratio is equal to zero) check for both the calculated and measured pressure patterns. At the rear of the sphere surface the theoretical pressures for a perfect fluid have the same positive values as for the corresponding area on the

front. The measured pressures on the rearward side of the sphere show that, instead of a region of positive pressures which would be present in a perfect fluid, there is a region of negative pressures. This discrepancy may be explained as follows: In a perfect fluid such as was assumed for the theory, the flow around any great circle of the sphere would be symmetrical. If this were so, the symmetrical pressure distribution diagram indicated by the dotted lines of Fig. 12 would be obtained and the sphere would have zero drag or resistance. In the imperfect fluid that air is, this ideal condition cannot be realized as the air moves over the sphere surface with viscous drag, with the building up of a boundary layer; ultimately, the tearing away of the boundary layer on the rear half; the creation of an eddying region; and, consequently, a drop in pressure behind the sphere. This actual flow is represented diagrammatically

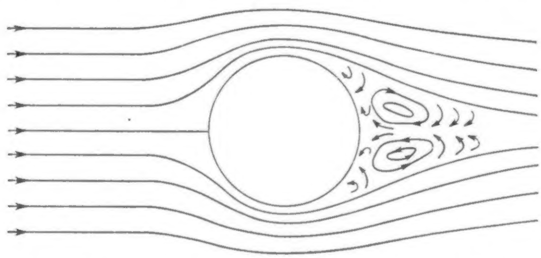


FIG. 15.—DIAGRAM OF ACTUAL AIR FLOW AROUND A SPHERE

in Fig. 15. It is interesting to note that the maximum suction developed experimentally is somewhat greater than the theoretically calculated value. This does not necessarily indicate that the tangential velocity is higher, since loss of pressure is experienced due to viscosity.

Sphere in Presence of the Ground.—The comparison of the theoretically calculated and measured pressures for the sphere in the presence of the ground board is shown in Figs. 13(*a*), 13(*b*), 13(*c*), and 13(*d*) in four plots so that all the great circles used are represented. The plots show that the theoretical and measured pressures agree very well for the part of all the great circles that is on the upper forward half of the sphere. In this region too, the points at which $\left(\dfrac{p_n - p_\infty}{\rho \dfrac{V^2}{2}}\right) = 0$ are in good agreement for both the theory and practice. On the rear half of the sphere the same general discrepancies are noted as were shown for the free air sphere. The most interesting points in the comparison occur on the lower side of the sphere that is in close proximity with the ground board. Here, it will be seen that the greatest difference between theory and measurement is shown for the meridian plane and the least for the equator plane. Upon following the general effects on all planes from the lower half of the meridian up to the equator, it can be seen that the discrepancies between theory and measurement tend to lessen progressively. The general conclusions that might be drawn from the measured pressure distribution diagrams are:

(1) Hydrodynamic theory indicates that, in a perfect fluid, the presence of the ground board should cause a greater tangential velocity over the entire

surface of the sphere, and not only on the lower part of the sphere near the ground board. A study of the suctions in Figs. 12, 13(*a*), and 13(*b*), indicates that experiment is in accord with the theory.

(2) On the lower half of the sphere, in the presence of the ground board, there is greater discrepancy between theory and experiment. A boundary layer is built up over the ground board within which the air is retarded. This is equivalent to a construction of the air passage between sphere and ground, and explains the much higher velocity on the lower part of the meridian plane. Figs. 13(*a*) and 13(*b*) indicate that the maximum suction attains relatively very high values. For points that are farther away from the direct influence of the ground this effect is lessened.

(3) The presence of the ground board intensified the eddying region at the back of the sphere, on its lower half.

(4) The generally greater tangential velocities (and, hence, greater skin friction), the far greater tangential velocities over the parts of the sphere nearest the ground, and the intensification of the eddying regions, confirm the results of the drag tests, which showed that the drag of the sphere in the presence of the ground board is far greater than in free air.

(5) From the pressure distribution diagrams it is clear that the transverse, or lift, force would be in a downward direction. This was shown to be the case by actual measurement, the results of which have been given in Table 2.

Sphere in Presence of the Ground and Collar, and in Presence of Ground and Columns.—The pressure distribution diagrams for a sphere in the presence of a ground board and a collar, and in the presence of a ground board and columns, are plotted in Figs. 13(*e*), 13(*f*), 13(*g*), and 13(*h*) for the meridian and equator planes only. These two planes or great circles were chosen as being the most representative. Study of the diagrams of Figs. 13(*e*), 13(*f*), 13(*g*), and 13(*h*) leads to the following conclusions:

(6) The introduction of either the collar or the column changes the character of the flow appreciably on the meridian plane. The positive pressures on the meridian plane at the windward side of the sphere now extend far below the equator. The introduction of either columns or collars impedes the flow, as would be expected, and hence creates the pressure region. The production of pressure below the equator explains the fact that the transverse force or lift on the sphere is now upward, as shown by the force tests.

(7) The introduction of both the collar and the columns disturbs the flow considerably and produces intensification of the eddying region at the back of the sphere, below the equator. This explains the production of a new region of suction which was not present in the case of the sphere in free air, or in the presence of the ground.

(8) At and above the equator, the pressure distributions in the presence of the ground and in the presence of the ground and collar, or column, are very similar. It is on the lower half of the sphere that the introduction of the collar or a column produces the most marked effects.

(9) The intensification of eddying and the production of a new region of suction when a collar or column is introduced explain, adequately, the greater drag of the sphere as compared with its drag in the presence of the ground alone.

(10) The introduction of the collar affects the drag and pressure distribution far more than the introduction of the columns, which still provide a passage for air flow close to the ground. From the point of view of structural loads, mounting on columns is preferable to mounting on a collar.

(11) The introduction of the collar or column produces great asymmetry of pressure on the meridian which the structural engineer would no doubt take into account, taking care to examine local effects.

Sphere on Collar with Trylon at Side.—The pressure diagrams of Figs. 14(*a*) and 14(*b*), illustrating the aforementioned condition, constitute a composite diagram for the equator plane, since only 180° of the equator plane was provided with orifices. In plotting the pressure at the equator plane the 180° on the Trylon side was obtained from actual observed data. On the far side of the equator plane away from the Trylon, data were taken from the test with collar but no Trylon, the assumption being that the presence of the Trylon would not appreciably affect the pressure distribution on the far side.

The introduction of the Trylon on the side does not cause marked variations in the pressure distributions. A twisting moment about the vertical axis of the sphere would be introduced, but this should not be serious.

Sphere with Collar and Trylon Ahead.—The pressure distribution for this case is illustrated in Figs. 14(*c*) and 14(*d*). The Trylon acts very powerfully as an aerodynamic shield. On the meridian plane pressure effects practically disappear, with all the orifices under a very slight degree of suction. The shielding effect on the equator plane is very marked. The pressure on the windward side of the sphere almost disappears, and the suction diminishes to a value of $\left(\dfrac{p_n - p_\infty}{\rho \dfrac{V^2}{2}}\right) = -1.0$ (approximately).

General Conclusions

From the point of view of the structural engineer some general conclusions appear to be of interest:

(12) For even a semi-stream-lined object, such as a sphere, it is impossible to design for air loads on the basis of some empirical rule for wind pressure. The same point of the structure may, at one time, attain a maximum pressure, and, at another time, a high suction value.

(13) Intense variation in pressure differences from point to point must be expected and allowed for in design.

(14) Instead of endeavoring to apply general rules, a systematic survey of pressure distribution is necessary, with subsequent integration of loads along planes in three directions, and integration of local load distribution. It is understood that Shortridge Hardesty, M. Am. Soc. C. E., has made use of the data herein presented for the most painstaking load integrations.

(15) The heaviest loads will be encountered when the Trylon is at the side of the sphere when the wind is considered as striking the nose of the sphere.

(16) The only points that will always be in high suction are those toward the upper surface of the sphere.

(17) The shielding effect of the Trylon should be taken into consideration with saving in weight of certain members.

(18) In general, structural members would have to withstand tension and compression loads under different wind conditions.

(19) The maximum positive pressure will be experienced at the equator and will never exceed $\left(\frac{p_n - p_\infty}{\rho \frac{V^2}{2}}\right) = 1.0.$

(20) Since the static pressure inside the sphere should be about equal to the outside static pressure it is inconceivable that pressures in excess of those shown in the pressure diagrams should ever exist. This being the case, and if 100 miles per hour is assumed as the maximum possible outside air velocity, then the maximum positive pressure on the sphere would be 25.6 lb per sq ft, and the negative pressure about −30.8 lb per sq ft on the sides of the equator plane.

Acknowledgments

The investigation reported herein was undertaken at the request of the Construction Department of the New York World's Fair Corporation. For their active co-operation, thanks are due especially to John P. Hogan and L. B. Roberts, Members, Am. Soc. C. E., Chief Engineer and Assistant Chief Engineer, respectively, of that organization. Thanks are also due to Mr. Hardesty for his critical guidance in the conduct of the experimentation. All the major and auxiliary apparatus in the laboratories of the Daniel Guggenheim School of Aeronautics, College of Engineering, New York University, was made available for this purpose.

APPENDIX

List of References

Some of the following references are cited in the paper. The remainder have a general significance in relation to the entire paper.

(1) "Standards of Design for Structural Steel," *Bulletin 12 Yb*, U. S. Navy Dept., September, 1934.

(2) "Speed and Pressure Recording in Three-Dimensional Flow," by F. Krisam, *Technical Memorandum, No. 688*, National Advisory Committee for Aeronautics.

(3) "Druckverteilung an der luftumströmten Kugel," by O. Krell, *Zeitschrift für Flugtechnik und Motorluftschiffahrt*, February 28, 1931.

(4) "Flow and Drag Formula for Simple Quadrics," by A. F. Zahm, *Technical Report, No. 253*, National Advisory Committee for Aeronautics.

(5) "Wind Stresses in Buildings," by Robins Fleming.

(6) The Code of Ordinances of the City of New York Building Code, Section 54.

(7) "Wind Pressure on a Model of the Empire State Building," by H. L. Dryden and G. C. Hill, *Journal of Research*, National Bureau of Standards, April, 1933.

(8) "Wind Pressure on a Model of a Mill Building," by H. L. Dryden and G. C. Hill, *Journal of Research*, National Bureau of Standards, April, 1931.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

RECLAMATION AS AN AID TO INDUSTRIAL AND AGRICULTURAL BALANCE

Discussion

BY ERNEST P. GOODRICH, AND CALVIN V. DAVIS, MEMBERS, AM. SOC. C. E.

ERNEST P. GOODRICH[25] AND CALVIN V. DAVIS,[26] MEMBERS, AM. SOC. C. E. (by letter).[26a]—In closing, the writers re-emphasize the existing needs for the economic re-adjustments outlined in this paper: (1) Technological advances are continuing to increase unemployment; (2) the workers thus displaced have, in most cases, no secondary means of support; (3) the overcrowded and restricted conditions in many industrial cities prevent any constructive use of idle time; and (4) the accumulated national loss resulting from this idle time is such a staggering value that almost any means, no matter how drastic, would be justified to prevent it. Consider, for example, a fair measure of this loss. At the nominal unit rate of $1 000 per worker the annual loss to the country, with 10 000 000 persons idle, would be $10 000 000 000. From a banker's viewpoint the prevention of this loss would be worth an investment of $100 000 000 000, or approximately three times the national debt. This is a challenge to engineers.

There are indications that the Medical Profession is already aware of the need for the social changes recommended by the writers. For example, Dr. Alexis Carrel has written:[27]

"There have been in the past industrial organizations which enabled the workmen to own a house and land, to work at home when and as they willed, to use their intelligence, to manufacture entire objects, to have the joy of creation. At the present time this form of industry could be resumed. Electrical power and modern machinery make it possible for the light industries to free themselves from the curse of the factory. Could not the heavy indus-

NOTE.—The paper by Ernest P. Goodrich and Calvin V. Davis, Members, Am. Soc. C. E., was published in November, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1937, by Joseph Jacobs, M. E. McIver, and C. S. Jarvis; April, 1937, by Messrs. John P. Ferris and L. C. Gray; and September, 1937, by Charles P. Williams, M. Am. Soc. C. E.

[25] Cons. Engr., New York, N. Y.

[26] Formerly, Chf. Engr., Ambursen Eng. Corp., New York, N. Y.

[26a] Received by the Secretary April 7, 1938.

[27] "Man the Unknown," by Alexis Carrel, p. 315.

tries also be decentralized? * * * Men would live in small communities instead of immense droves. Each would preserve his human value within his group. Instead of being merely a piece of machinery he would become a person."

The writers are in accord with Mr. Jacobs and Mr. McIver that this entire matter should be approached from an engineering angle. Engineers should be willing to assume a large measure of responsibility during the transition period.

The suggestions and analyses presented by the writers were not submitted as a possible cure-all for the problems outlined in the foregoing; these were intended to show by one specific example the possible benefits that could be realized from correlating the conservation of natural resources with industrial and economic adjustments. Conservation programs, such as are now (1938) under way in the arid regions of the West, could have as their larger objective the ending of unemployment. The investment in each project, in a sense, could be considered as an increment of the total that would be required to capitalize the time now being lost during depressions. Mr. Williams makes a point in this respect by indicating the over-all economic effects of the projects built by the U. S. Bureau of Reclamation even if many of these projects would not be commercially feasible investments when considered individually. Add to these existing benefits the partial elimination of unemployment, or, as Mr. Williams prefers to call it, furnishing a supplementary means of income during slack periods, and the accumulated benefit values will be many times the project costs.

Improvement, of course, cannot be brought about quickly; it will be rather a matter of slow growth. A long period of preparation, co-ordination, and training will be required, as suggested by Mr. Jarvis. It is unlikely that industrial growth with parallel colonization and agricultural development, as suggested by Mr. Gray; the writers are of the opinion that factories will follow only after the country is fairly well developed.

The writers intended to emphasize the need of a better balance between agriculture and industry in a given area rather than complete self-sufficiency as inferred by Mr. Gray and Mr. Jacobs. As Mr. Ferris so ably stated, there has been a mining process in some areas which has drained them of both human and material resources. In some cases, decentralization will help to restore, and in others establish, a more balanced economy.

In conclusion, the writers express their appreciation for the interesting and timely discussions that were contributed.

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DISCUSSIONS

CONSTRUCTION AND TESTING OF HYDRAULIC MODELS MUSKINGUM WATER-SHED PROJECT

Discussion

By George E. Barnes, M. Am. Soc. C. E., and J. G. Jobes, Jun. Am. Soc. C. E.

George E. Barnes,[12] M. Am. Soc. C. E., and J. G. Jobes,[13] Jun. Am. Soc. C. E. (by letter).[13a]—The discussion shows corollary experience in other laboratories, with the inherent difficulty of securing desired precision in piezometer and velocity readings. The value of model studies, now established beyond question, will be much enhanced, of course, by any improvements in laboratory technique.

Messrs. Howard and Edwards both submit practical suggestions on the piezometer connections, piezometer tubing, and indicating devices. With these, and with Mr. Edwards' comments on velocity meters, the writers concur. In the laboratory at Case School of Applied Science, hook-gage buckets for gangs of piezometers have been found less practical, in most cases, than a bank of piezometer-indicating tubes joined at the top by a header, and connected to pressure and suction lines by suitable valves. It all depends, however, upon the precision and convenience desired, and no generalization is offered.

As the writers stated, too much confidence is not warranted in the experimental values of n determined for these short model tunnels. Professor Powell amplifies this observation by reference to the probable influence of the intake structure on velocity distribution. In a thesis study,[14] it was demonstrated conclusively that the apparent value of n in the model tunnel of Pleasant Hill

Note.—The paper by George E. Barnes, M. Am. Soc. C. E., and J. G. Jobes, Jun. Am. Soc. C. E., was published in December, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1937, by Messrs. G. W. Howard, F. W. Edwards, and T. T. Knappen; and September, 1937, by Ralph W. Powell, M. Am. Soc. C. E.

[12] Head, Dept., of Civ. Eng., and Prof., Hydr. and San. Eng., Case School of Applied Science, Cleveland, Ohio.

[13] Engr., U. S. Engr. Office, Los Angeles, Calif.

[13a] Received by the Secretary April 14, 1938.

[14] "Determination of Kutter's *N* for the Tunnel of the Pleasant Hill Dam Model," by Edward R. Van Driest, Harold E. Orford, and Charles M. Gard. Thesis submitted for the degree of Bachelor of Science in Civil Engineering, Case School of Applied Science, 1936.

Dam could be changed by baffling the entrance in a manner that would influence velocity distribution. Furthermore, the measurements indicated that the value of n may change with the rate of discharge; and, as implied by Mr. Edwards, a value of n which is on the safe side for the capacity of the tunnels, is on the unsafe side for the performance of the stilling-basin. It is necessary to determine the value of n in order to calibrate the delivery capacity of the outlet works. In operating the stilling-basin, however, it is the quantity of discharge that is important. For a given discharge, excess head may be needed.

All observers agree that the design must provide for probable deviations in tail-water level from those predicted. The extent of this deviation is a matter of judgment based on the probable strength of the data from which the tail-water rating curve is derived.

Mr. Knappen's discussion is most generous in scope. As the then Chief of the Engineering Division of the U. S. Engineer Office, at Zanesville, Ohio, his viewpoint is most valuable. Perhaps too much weight is given the dollars and cents savings made possible by model studies and too little weight is given the fact that such studies, in many cases, are worth while simply as a guaranty of good performance. Certainly, the art of stilling-basin design alone was greatly advanced in this work, and as Mr. Knappen indicates, it became more and more possible to be right the first time as the studies progressed. This was despite the wide variety of designs for the several dams.

The design of the Clendening Dam, discussed by Mr. Knappen, was mentioned by the writers as an interesting departure from orthodox design. It also illustrates how an idea for design will germinate during the manipulation of a model. In previous and incompleted tests on Tappan Dam, stilling-basin performance with the tunnel at various elevations was studied for low discharges by merely inserting filler pieces or "humps" on the floor of the tunnel portal, as described. This was quicker and more convenient than continually changing the model around and, in a rough and tentative manner, it gave the approximate effect of raising or lowering the tunnel. Here was born the idea of a low tunnel, with a portal or apron of greater lateral width and high invert, although a simpler design was preferred if field conditions would permit raising the tunnel throughout its length. However, the so-called "hump" proved well adapted to field conditions at Tappan, Clendening, and Piedmont Dams, and the first design for Clendening was so drawn up, although some changes were made after test.

Professor Powell dwells on the expedition with which the work was handled, and the limitations which this imposed on the scope of the tests. Expedition marked the handling of the Muskingum Water-Shed Project as a whole, and was, in fact, a condition precedent to the approval of the project. It might be stated, therefore, that the model studies would have failed in their purpose, had each individual study been extended beyond the time allowed. Unquestionably, this program limited the findings secured.

The writers did not intend to elaborate upon technical findings and design, but rather on matters affecting the programming of model studies. The following remarks are offered in closure: As part of the design procedure, model

studies for a given project may be simple or elaborate depending on objectives. These objectives may be limited to taking observations on matters of only immediate or primary concern in the design, or they may be extended in detail or expanded to general research to facilitate the planning of a series of integrated developments with other projects in view. Model studies ordinarily should start with preliminary studies and should be carried along concurrently with design. The writers believe it undesirable to issue contract drawings for bidding purposes before the model studies are completed, on the assumption that changes will be minor in character, since under these circumstances they are likely to be in the nature of a compromise and the design does not receive the full benefit of the model studies. In the case of the Muskingum Water-Shed Project, model studies commenced at the very beginnings of design, which is the best possible arrangement, and most productive for all concerned.

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DISCUSSIONS

NATIONAL ASPECTS OF FLOOD CONTROL
A SYMPOSIUM

Discussion

BY MESSRS. WILLIAM F. UHL, ARTHUR W. HARRINGTON AND HOLLISTER JOHNSON, AND MERRILL BERNARD

WILLIAM F. UHL,[74] M. AM. SOC. C. E. (by letter).[74a]—To the many engineers who have contributed discussion to his paper, the writer is indebted. Mr. Williams' discussion of the basis for computing flood damages, which he illustrates by reference to the Winooski River, is helpful. The writer agrees with Mr. Williams that the humanitarian aspect of flood control should not be stressed to the point of causing a community to assume a burden which threatens its financial ability.

By his account of the flood at Lowell, Mass., Mr. Safford has described the effect of the greatest flood on record on one of the old water-power cities of New England. The damage that occurred, and the problem of flood control as it affects Lowell is probably typical of the situation in New England cities situated on the larger rivers. In his study of flood frequency on the Lower Connecticut River, Professor Barrows has made use of one of the longest records of flood stages in the United States—that of the Connecticut River, at Hartford, Conn. The fact that he has developed two extension curves from these data illustrates the difficulty of establishing an accurate basis for computing the frequency of floods of certain magnitudes.

Mr. Bush questions whether it is possible to operate reservoirs, built for the dual purpose of power storage and flood control, in such a way that the flood-control aspect will be fully safeguarded. In such a dual-purpose reservoir, un-

NOTE.—This Symposium was presented at the Fall Meeting of the Society and at the meeting of the Waterways Division, Pittsburgh, Pa., October 13 and 14, 1936, and published in March, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: June, 1937, by Messrs. F. C. Scobey, Howard T. Critchlow, T. T. Knappen, M. C. Tyler, Gordon R. Williams, Arthur T. Safford, W. G. Hoyt, J. D. Arthur, Jr., John H. Meursinge, H. K. Barrows, E. D. Hendricks, and Edward W. Bush; September, 1937, by Messrs. H. K. Barrows, Ivan E. Houk, and John E. Field; October, 1937, by Messrs. C. S. Jarvis and Joseph Jacobs; December, 1937, by Messrs. W. M. Dawley, and Howard M. Turner; January, 1938, by Ralph W. Powell, M. Am. Soc. C. E.; and April, 1938, by Franklin F. Snyder, Jun. Am. Soc. C. E.

[74] Hydr. Engr., Chas. T. Main, Inc., Boston, Mass.

[74a] Received by the Secretary March 26, 1938.

questionably some of the effectiveness of a given reservoir for power-storage purposes will have to be sacrificed to the requirements of flood control. In New England, at the present time, there are several instances of power-storage reservoirs subject to regulations dictated by the recreational uses of the reservoir, which impair their effectiveness as power-storage reservoirs; for instance, the operation of Lake Winnepesaukee is subject to State regulations which limit the draft during the summer months. It is a fact that the recreational uses of these reservoirs have been safeguarded by these regulations. Therefore, it seems reasonable to suppose that, in the case of reservoirs built for the dual use of power storage and flood control, it would be possible to control the operation by regulations which would safeguard the requirements of flood control.

The writer agrees with Mr. Turner that the isohyetal lines of very high rainfall in the second storm, as shown in Fig. 5, probably did not extend as far west as is shown. An exact determination of the area covered by the 10-in. and the 8-in. rainfall is an interesting subject of speculation, but there are not sufficient data available to arrive at a definite answer to the problem. Mr. Turner's study of the major peak of the 1936 flood on the Connecticut and Merrimac Rivers [66] is a valuable contribution to the subject of New England floods.

The writer wishes to point out that his paper was written while the work of collecting and analyzing the data on the March, 1936, floods was still in process. As a result, many of the figures given in the paper, particularly those of 1936 peak discharges, are subject to change. In general, the changes are so small that it seems unnecessary to revise the original data. The revised values are available in some of the more recent papers on this subject.[75]

ARTHUR W. HARRINGTON,[76] M. AM. SOC. C. E., AND HOLLISTER JOHNSON,[77] ASSOC. M. AM. SOC. C. E. (by letter).[77a]—Since the presentation of the writers' paper, the U. S. Geological Survey has completed a number of papers and reports pertinent to the subject of this paper. One series, of three parts, concerns "The Floods of March 1936." [78] In these volumes are presented records of stage and discharge at river-measurement stations, contents of storage reservoirs, peak discharges and stages with comparative data for other floods, crest stages, precipitation, the depth and water content of snow on the ground, the results of studies of rainfall and run-off, and many other kinds of flood information.

The writers are grateful to Mr. Hendricks for his discussion of the floods of 1935 and 1936, with special reference to their occurrence and relation to previous floods in the basins of the Mohawk and Oswego Rivers.

Professor Powell has a very interesting idea of plotting drainage area against the discharge per square mile for the maximum annual floods over the entire period of record of a large number of streams and representing the 100-yr prob-

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[66] *Journal*, Boston Soc. of Civ. Engrs., Vol. XVII, No. 7, September, 1930.

[75] See, especially, "The Floods of March 1936, Part I, New England Rivers," *Water Supply Paper 798*, U. S. Geological Survey.

[76] Dist. Engr., U. S. Geological Survey, Albany, N. Y.

[77] Hydr. Engr., U. S. Geological Survey, Albany, N. Y.

[77a] Received by the Secretary February 21, 1938.

[78] "Part 1, New England Rivers," *Water Supply Paper No. 798;* "Part 2, Hudson River to Susquehanna River Region," *Water Supply Paper No. 799;* and "Part 3, Potomac, James, and Upper Ohio Rivers," *Water Supply Paper No. 800.* (Publication pending.)

ability by a line or smooth curve which would have 1% of the points outside it; the 20-yr probability by the line with 5% of the points outside it, etc. However, some difficulties might be encountered in establishing a criterion by which to judge the distribution of the points along the line and thus determine the slope of the line. If some reasonable criterion was not used it would be easily possible to produce an absurd interpretation that would show increases in unit discharges with increases in drainage areas which, of course, is generally not true. It is the writers' belief that, if, before plotting, the discharges per square mile were corrected for channel storage (thus representing the unit rate of run-off into the channel rather than the unit rate of run-off in the channel past the measuring point) the resulting chart would represent more nearly unit rates of run-off which might be expected to occur in streams in the same locality but with perhaps much less channel storage.

It appears that there is unanimity of opinion among engineers and others as to the need for the development of a national flood-protection policy and that one of the essentials for the evolution of such a policy is the availability of the hydrologic data which are so basic and fundamental in any study and investigation of the control of water. Including the aforementioned series,[78] the U. S. Geological Survey has prepared reports on ten notable and outstanding floods which have occurred since 1934. These reports[79] were planned as an effort to present such basic hydrologic data (especially that of precipitation and stream flow) as were available at the time of publication.

MERRILL BERNARD,[80] M. AM. SOC. C. E. (by letter).[80a]—It is gratifying to report that field headquarters for five of the hydrologic regions planned by Mr. Hayes were established early in 1938. Other gains in expanding and strengthening the service have been made, one of the most important being a co-operative program between the Commonwealth of Pennsylvania, through its Water and Power Resources Board, the U. S. Weather Bureau, and the U. S. Geological Survey. This co-operative project is providing the means to establish a modern forecasting service on the Allegheny, Monongahela, Susquehanna, and Delaware Rivers.

Modern methods of estimating stream flow from rainfall extend the analysis into the relatively small water-sheds comprising the river basin. The procedure involves short concentration periods, and rainfall rates of short duration and high intensity. Obviously, rainfall rates recorded as averages for 24 hr cannot be used in studies on water-sheds of 50 to 500 sq miles having concentration periods of 2 to 6 hr. Therefore, the first step in the aforementioned co-operative program has been to establish, over the river basins comprising the project, a

[79] "Flood in the La Canada Valley, California, January 1, 1934," *Water Supply Paper No. 796–C;* "Flood on Republican and Kansas Rivers, May and June, 1935," *Water Supply Paper No. 796–B;* "The New York State Flood of July, 1935," *Water Supply Paper No. 773–E;* "Major Texas Floods of 1935," *Water Supply Paper No. 796–G* (includes the floods of May, June, and December) (publication pending); "Major Texas Floods of 1936," *Water Supply Paper No. 816* (includes the floods of June, July, and September); "The Ohio-Mississippi Floods of January–February, 1937" (publication pending); and "Major New Mexico Floods of 1937" (publication pending).

[80] Mr. Hayes died on November 16, 1936. This closing discussion on his paper was prepared by Merrill Bernard, M. Am. Soc. C. E., who succeeded Mr. Hayes as Chf. of the River and Flood Div., U. S. Weather Bureau, Washington, D. C., on March 5, 1937.

[80a] Received by the Secretary March 17, 1938.

net of approximately 130 recording rain-gages having an average density of 400 sq miles per gage.

These recording rainfall stations have been classified as: (1) Those that are located primarily to provide data for the hydrologic studies from which forecasting methods are to be developed; and (2) those that are to serve ultimately as reporting rainfall stations under the established forecasting procedure. Stations in the first group have been located so as to give proper weight to geographic position in the net, area representation, elevation, and aspect, without depending upon local attendance. Those in the second group have been located at strategic points where qualified 24-hr attendance is available and dependable communication assured.

Practically all the rainfall stations are accessible over paved highways and have been located on three circuits over which experienced hydrologic engineers travel on schedule. These engineers are charged with the duties of servicing the gages, assembling the charts, and tabulating the data for immediate use. During the winter months they will conduct snow surveys and keep themselves informed of day-by-day changes in run-off conditions.

In general, the procedure at the forecasting centers will be as follows:

(*a*) A day-by-day estimate of run-off potentialities based upon the discharge from index water-sheds, the estimate to include the possible contribution to run-off from an existing snow mantle;

(*b*) A day-by-day check on storm approach, utilizing the general weather service of the Weather Bureau, and particularly the weather maps prepared at frequent intervals at the aerological stations;

(*c*) With the beginning of a critical storm period, the reception of hourly (or other-period) rainfall by telephone and telegraph, or by amateur and automatic radio broadcasting;

(*d*) The preparation of an isohyetal map for each period showing the progressive accumulation of rainfall depth over the basin;

(*e*) Under a well organized and largely mechanical technique, the conversion of rainfall into stream-flow hydrographs;

(*f*) The conversion of rates of stream flow to river stage at points down stream for which a forecasting service is rendered; and,

(*g*) As quickly as rainfall has ceased, the prompt announcement, through established channels, of flood-peak stage and the hour of the arrival of the flood's crest.

Mr. Snyder has given an interesting picture of the March, 1936, flood at Pittsburgh, Pa., based upon a critical review of the meager data available and an application of the unit graph method of estimating stream flow.

The unit hydrograph is found to be an excellent tool for forecasting, as his discussion indicates. As data are made available through the newly established network of recording rainfall gages, refinements in the method and time-saving devices in procedure can be expected to develop. His study constitutes an important step in the establishment of a modern forecasting plan for Pittsburgh.

In closing the incident of the forecasts of the March, 1936, flood at Pittsburgh, it seems appropriate to comment upon the courage which it must have taken to issue an unprecedented forecast based upon the most meager data, and dependent upon a method in which judgment gained from personal experience was the predominating factor.

Mr. Scobey has called attention to the important place snowfall has in the genesis of floods, particularly on Western streams where flow is comprised largely of run-off from snow melting.

In the mountainous States of the West forecasts of the seasonal yield of the snow mantle, expressed as a percentage of the normal for the basin, are determined from surveys made toward the end of the snow-fall season. The obvious need for a quantitative forecast—that is, a forecast in terms of the hydrograph of flow—has stimulated an interest in the Weather Bureau's investigations of snowfall gages, the most promising type being one in which the snow is melted as it falls into the gage with a salt brine, the catch stabilized against wind action with a flexible shield, and the contents of the gage protected against evaporation by an oil film.

The snow course lends a real significance to the record by virtue of its length. Likewise, the snow-fall stations of the Bureau will utilize sampling areas, carefully selected as typical, upon which are placed batteries of storage snow gages, the average record of the group being considered applicable to the part of the basin represented by the snowfall station. The average water depth for the group (usually five gages) is determined by the simple expedient of weighing, under which procedure the increment increase in weight between weighings becomes the snowfall for the period, expressed in inches. Batteries of such gages have been established at twenty stations having elevations greater than 7 000 ft. Gratifying experience to date (1938) indicates that they have the following advantages over other methods of determining the water equivalent of snowfall under Western conditions: (1) They measure snow as it falls and before it is subjected to prolonged wind action; (2) they secure the record against loss through poor attendance or failure to reach the station before a critical period of melting; (3) they function at all elevations, including the transition zone from snowfall to rainfall; (4) they are inexpensive to install and require a minimum of attendance; (5) they are not dependent upon tree shelter for stability of record; and (6) precipitation in the form of snow can be recorded as daily precipitation through the simple procedure of daily weighing.

One hesitates to take up the gauntlet in Mr. Meursinger's good-humored jibe at American engineers for their temerity in attempting to determine possible maximum run-off of the Ohio and Mississippi Rivers. Such optimism has evidenced itself in the newly established hydro-meteorological studies of the Weather Bureau set up co-operatively with the U. S. Corps of Engineers. The study undertakes to provide the meteorological and hydrological background of the great storms of record and the justification for their transposition to basins other than those embraced actually by the critical storm area.

Mr. Houk has sensed the weakness in any head-water forecasting service to meet the situation developing around the highly intense cloudburst type of

storm which can develop locally and may not be associated with a period of general rainfall.

Two circumstances may mitigate toward a practical solution of such forecasting problems:

(a) *Plans for an Expanded Network of Recording and Reporting Rainfall Stations.*—The density of the co-operative network in Pennsylvania is approximately 1 gage to 400 sq miles. Recent estimates for networks on problem areas in the Mississippi Basin provide a recording rain-gage to each 100 sq miles on the average.

(b) *The Rapid Expansion of the Network of Aerological Stations of the U. S. Weather Bureau.*—These stations, located at the active airports of the United States, serve aviation and issue a weather map at 6-hr intervals from transmitted signals which include the results of upper-air soundings.

It is planned to make full use of these aerological stations in the river and flood forecasting service of the Weather Bureau.

Head-water flood-warning services are being inaugurated (1938) by the Bureau in co-operation with the Cities of Cumberland, Md.; Johnstown, Pa.; and Elkins, W. Va. In these instances no immediate attempt is being made to forecast flood stages, but a carefully organized warning service is intended which will utilize municipal departments, railroad and utility company communication facilities, and the services of municipal officials and employees whose interests insure an alert and efficient attendance.

The snow work of the Weather Bureau has been commented upon briefly in the discussion by Mr. Scobey. Mr. Field has emphasized the need for quantitative forecasts of run-off from melting snow. Under conditions in the Central and Eastern States, where periodic melting may remove the snow mantle several times during the winter months, it is necessary to know, day by day, the potential run-off in an existing snow cover, and an adequate forecasting system must provide for the measurement of the factors that may combine to release such run-off. Under Western conditions, where there is no appreciable loss through the winter, the problem of quantitative forecasting will be largely that of measuring factors, such as temperature, concurrent rainfall, and ground surface conditions, as well as snowfall measurements at a sufficient number of points in a basin to give dimensional significance to the average depth of snow and water equivalent. The gathering of such data involves remote observational points at high elevations where attendance is difficult or impossible and emphasizes the obvious need for automatic recording seasonal snow gages.

The greatest obstacles to be overcome in the reorganization and expansion of the Bureau's river and flood forecasting service is a general lack of understanding of its functions, inadequate financing, and the difficulties of adjusting cost to benefit. The responsibility as well as the expense of a flood-forecasting service should shift in varying degrees from the Federal Government only, to a co-operation with other interests as head-waters are approached and as problems become more and more localized. This transition in economic responsibility is accompanied by marked changes in method. Thus, in flood control, levee systems only give way to combinations with reservoirs and organized

land-use practices over large areas. Likewise, methods of river forecasting which have met successfully the conditions of lower river flow must be elaborated to take into account the many factors influencing smaller stream flow, the principal one of which is rainfall.

On the larger river systems the objective for the service is an integrated forecast from head-waters to outlet. This is best understood by conceiving the principal channel of the river as divided into reaches of known channel storage. It can be stated, then, that the discharge from the lower end of any reach throughout a fixed interval of time is equal to the discharge into the upper end from the reach above, plus the inflow of tributary streams entering throughout the length of the reach, plus or minus any change in storage that has occurred within the time interval. The travel of the flood wave through the reach must be taken into account. This takes the form of a progressive distortion in the shape of the storage prism throughout length and through time, and, under fixed conditions of inflow and outflow, may prove to be fairly constant. The most effective method of forecasting on a large river such as the Mississippi must begin at the head-waters, and on all important tributaries simultaneously, and must provide the means of transmitting hydrologic data and estimates of flow progressively down stream to the important forecasting centers along the main river stem.

The average annual flood loss to the nation as reported to the Weather Bureau is about $100 000 000. It is believed that this sum can be reduced materially by increasing the period of the forecast so that greater opportunity is provided for the removal of perishable goods and property and for the evacuation and protection of areas doomed to inundation by the oncoming flood. The cost of providing the additional preparation opportunity will prove negligible when compared with the resulting reduction in loss of life, human misery, and property damage.

The answer to the problem of minimizing national flood hazard does not lie in the permanent evacuation of the flood plains of the great rivers. Man has profited greatly by utilizing these fertile lands during such time as the rivers have not needed them to pass their flood waters to the sea. He should, and does, however, pay a rental for his occupancy, either in the continuing cost of protection works or in the periodic repair of flood damage. This cost could be reduced and, in many cases, nearly eliminated through a co-ordinated program of flood protection, flood warning, and flood insurance—all within the realm of practicability, as demonstrated in the several European countries pioneering in this field.

As the modern automatic fire alarm system is to be credited largely with the immunity now enjoyed from the catastrophic conflagrations that occurred all too frequently through the past half century, so should a modernized flood-forecasting service pave the way to a more secure and intelligent use of the river valleys of the United States.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

THE PASSAGE OF TURBID WATER THROUGH LAKE MEAD

Discussion

By Nathan C. Grover, M. Am. Soc. C. E., and Charles S. Howard, Esq.

Nathan C. Grover,[48] M. Am. Soc. C. E., and Charles S. Howard,[49] Esq. (by letter).[49a]—Since the publication of this paper interest in density currents has been manifested through correspondence, published discussions, and the creation of a specific committee (Interdivisional Committee of the National Research Council on Density Currents) for their further study. The discussions have served to emphasize the inadequacy of information with respect to the phenomena of density currents and the probable theoretical and practical value of further study of them. Several interesting and important aspects have been presented. In general, the discussions have disclosed: That there have been instances of flows of turbid water through reservoirs under a variety of conditions that cannot be evaluated because of lack of observational information; that there may be many instances of stratification of water in reservoirs due to differences in density, in many of which turbidity is not involved; and, that there is an important and unexplored field of scientific research in relation to density currents and stratification in reservoirs which may have both scientific and practical significance.

Mr. Faris has described a flow of turbid water through Lake Kemp and mentioned the presence of silt in depressions in Medina Reservoir. Messrs. Page and Forester have given valuable information concerning the visibility of the flow of the turbid water through the center of Lake Mead during the period before the depths of water were too great. Messrs. Faris, Dobson, and

Note.—The paper by Nathan C. Grover, M. Am. Soc. C. E., and Charles S. Howard, Esq., was published in April, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1937, by Messrs. O. A. Faris, Paul A. Jones, Carl E. Scofield, and Ivan E. Houk; September, 1937, by Messrs. William P. Creager, Harold K. Palmer, Morrough P. O'Brien, John C. Page, John H. Bliss, and B. H. Monish; October, 1937, by Messrs. D. M. Forester, A. D. Lewis, G. C. Dobson, and William W. Rubey; November, 1937, by Messrs. J. C. Stevens, and C. S. Jarvis; and April, 1938, by Messrs. R. E. Redden and Raymond A. Hill.

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[49] Chemist, U. S. Geological Survey, Washington, D. C.

[49a] Received by the Secretary April 15, 1938.

Lewis have described similar flows of turbid water through other reservoirs and discussed conditions of deposition in reservoirs which are indicative of other such flows which may not have carried through to the places of discharge. Messrs. Jones, Scofield, Creager, Bliss, and Stevens have discussed the conditions under which the flows have occurred in Lake Mead and other reservoirs and Mr. Palmer has suggested the similarity of the conditions causing turbid flows and those of lack of mixing or of stratification of fresh and salt water. Mr. Houk and Professor O'Brien have discussed the phenomenon on the basis of temperature differences. Mr. Monish has suggested a modification of Prandtl's expression for the criterion of mixing, and Mr. Rubey has made several computations to show the combined effects of the quantities and nature of the suspended matter, and the salinity and temperature of the water. He has also considered the effects of variations in velocity as it was measured at the Grand Canyon gaging station about 160 miles above the head of the lake.

Mr. Jarvis has referred to earlier considerations of the subject and has suggested the practicability of creating turbulence as a means of putting the silt into suspension for passage out of the reservoir. The information presented by Mr. Lewis in his discussion and in the paper to which he refers,[33] is very valuable in outlining the efforts that have been made to eliminate silt from certain reservoirs in South Africa.

Mr. Page has presented additional information concerning conditions in Lake Mead during certain periods when the flows of turbid water took place. On the information available, the writers do not now agree with Mr. Page that the higher temperature of the relatively warm water near the bottom of the lake is explained by the warm springs known to be present near the dam site. The source of the heat producing this higher temperature in the lake water is not positively known, but water of the observed temperature appears to the writers to have extended over too large an area to be considerably influenced by the relatively small discharge of water from warm springs. It may be significant that the temperature of this stratum of warm silty water is 68° to 70°F which is approximately the mean annual temperature at and near Boulder Dam.

The writers believe that the submerging or covering of certain areas mentioned by Mr. Jones was of little or no importance in causing the silt flows since the material carried past Willow Beach during the flows was much finer than would be expected to result from sliding or scouring in those areas.

Mr. Monish has discussed the extent of mixing that might be expected under different conditions of flow and suspended matter. He has computed limits for the critical values of mixing which seemed to hold for the flows of turbid water observed at Lake Mead in 1935, and has shown that these values might not be the same for another year, nor for another reservoir. He has computed his values from records obtained at the Grand Canyon station which were the best available to him. Such computations, based on records of water and silt that actually came through the reservoir, would be of interest since it appears to be reasonably certain that the sizes and conditions of flocculation of the

[33] Communication No. 5, to the Second Congress on Large Dams, Washington, D. C., 1936, entitled "Silting of Four Large Reservoirs in South Africa."

particles are of importance in their effect on rates of settling. Mr. Rubey has made a careful study of the available data and was not able to explain the flows of turbid water by consideration of densities or density differences, or by the densities and velocities of the water measured at the Grand Canyon gaging station. He has emphasized the need for observations on the degrees of flocculation and the velocities or slopes of the turbid water within the reservoir. Mr. Stevens has suggested that an agency or board be created to co-ordinate investigations of the Colorado River Basin, and that the results of comprehensive studies in the Colorado River Basin would have value in connection with studies of the Rio Grande and other silt-bearing streams of the arid West. Professor O'Brien has suggested that some of the solid particles were first carried to the bottom and, later, were resuspended as a result of turbulent flow and then were carried through the lake in the turbid flows. Mr. Lewis has suggested that the matter in solution causes flocculation during periods of turbid flow, which, in turn, causes bottom flow.

Mr. Hill has presented tables and graphs to show that the water flowing out of Lake Mead during part of 1935 was a mixture of the water that entered during the spring flood with other water in the lake. He has raised the question: "Will the normal release from Lake Mead be materially more saline than the weighted average inflow and will the best water be wasted at the time of spill or release for flood control?" Mr. Hill has also suggested that the passage of the more concentrated water through the lake may be of greater practical importance than the passage of turbid water.

Mr. Redden described a flood on the Weber River in Utah, in August, 1934, after which turbid water passed through Echo Reservoir, and stated that although the river often carries considerable suspended matter the water discharged at such times was usually clear, and that of many known instances when silt-laden waters appeared at the outlet gates of reservoirs in Utah, the phenomenon is more noticeable on some rivers than on others.

Samuel B. Morris, M. Am. Soc. C. E., has furnished, by personal letter to the writers, a general description of some observations made on Morris Reservoir, in California:

"The Morris Dam completed in 1934 creates a reservoir in San Gabriel Canyon 20 miles east of the City of Pasadena. The reservoir is four miles long and of maximum depth of 250 feet. Storage capacity is 38,360 acre-feet and surface area 417 acres. Following winter storms the water in the reservoir clarifies and becomes stratified. Temperatures at the bottom of the reservoir will generally be 50 to 55 degrees while surface temperatures will be 5 to 10 degrees warmer by April. In the spring of 1935 a flood discharge of warm water entered the reservoir and flowed along the bottom to the flood release outlets at the base of the dam discharging a muddy stream while the surface water in the reservoir remained clear. After two or three days the reservoir became cloudy throughout and of constant temperature.

"Apparently the warm silt-laden flood water was heavier than the colder water in the reservoir. After two or three days, however, sufficient of the silt was precipitated to cause the warm water to be lighter in specific gravity than the cool water above it. This caused the warm water to rise and the reservoir 'turned over' thoroughly mixing its contents and giving a constant temperature throughout. This is similar to the condition which usually occurs in the late fall of each year when the surface water becomes cooler than the deeper water."

Since the publication of the paper, two reports concerning the delta of the Colorado River[50] have appeared. These reports present considerable information on the nature of the silt carried by the Colorado River, which will be of value in future studies of silt movement.

In the spring of 1937 the National Research Council appointed an Interdivisional Committee on Density Currents which may serve as the co-ordinating agency suggested by Mr. Stevens. Meetings of this Committee were held in June, 1937, to formulate a program of field and laboratory studies. Intensive programs were planned for the Elephant Butte Reservoir and Lake Mead. During the summer of 1937, observations were made at the Elephant Butte Reservoir, but there have been no apparent movements of turbid water through the reservoir during that year. The results obtained during the year show variations in concentration of the water in different parts of the reservoir, but the complete data are not available to show the extent or the nature of the stratification.

Samples of water and records of its temperature were collected in August and September, 1937, in co-operation with the U. S. Indian Service, on the Gila and San Carlos Rivers above the San Carlos Reservoir and at the outlet of Coolidge Dam. Although some of the samples of the inflowing water had more than 1% of suspended matter (of which a large part consisted of particles with diameters less than 20 microns), no suspended matter passed through the dam during this period. The mean daily discharge of the San Carlos River during the period was less than 100 cu ft per sec for all but four days. The flow of the Gila was somewhat larger, and in the two months of the study more than 250 000 tons of suspended matter was carried past the Calva gaging station at the head of the San Carlos Reservoir. The failure of turbid water to pass through the reservoir may have been due to the small quantity of suspended matter and the small flow.

No turbid water has passed through Boulder Dam since the gates at the bottom of the reservoir were closed on May 1, 1936. Periodic observations have been made by the U. S. Bureau of Reclamation to ascertain the temperature and position of turbid water in different parts of the lake. Most of these observations have been made in the canyon section immediately above the dam, but, since the summer of 1937, additional observations have been made in Boulder Canyon, 20 miles above the dam, and in Virgin Canyon, 50 miles above the dam. The observations show that there has been a body of turbid water at the bottom of the lake, as mentioned by Mr. Page. Temperature observations made as a part of the sampling program show a marked increase in the temperature of the water at the points of transition from clear to turbid water. Samples of the turbid water show that the quantities of solid material increase with the depth. The bottom samples have been so thick that they could be poured into a bottle only with difficulty. Mechanical analyses of the suspended material taken from the bottom of the lake at several places show

[50] "The Colorado Delta," by Godfrey Sykes, *Publication 460,* Carnegie Inst. of Washington; pub. jointly by the Carnegie Inst. of Washington and the Am. Geographical Soc. of New York, 1937; also, "Delta, Estuary, and Lower Portion of the Channel of the Colorado River, 1933 to 1935," by Godfrey Sykes, *Publication 480,* Carnegie Inst. of Washington.

that the material consists chiefly of very fine particles, more than 90% being less than 20 microns in diameter. In July, 1937, there was about 20 ft of the silty water in Virgin Canyon and about 35 ft in Boulder Canyon. Observations made in Virgin and Boulder Canyons, in September and November, 1937, and in January, February, and March, 1938, showed varying quantities of turbid water in both canyons.

On the basis of several observations made at different parts of the lake Table 10 has been prepared showing approximate elevations of the lake bottom and of the top of the silt layer during the period July 1, 1937, to March 12, 1938.

TABLE 10.—ELEVATION OF LAKE BOTTOM AND TOP OF SILT LAYER AT SEVERAL POINTS IN LAKE MEAD DURING THE PERIOD, JULY, 1937, TO MARCH, 1938
(Surface of Lake Approximately 1 100 Ft Above Sea Level During the Period)

Period	Pierces Ferry		Virgin Canyon		Virgin River, Lower Narrows		Boulder Canyon		Black Canyon			
									Cape Horn		Boulder Dam	
	Silt layer	Lake bottom	Silt layer	Lake bottom	Silt layer	Lake bottom	Silt layer	Lake bottom	Silt layer	Lake bottom	Silt layer	Lake bottom
1937:												
July 1	....	....	....	....	....	....	....	....	....	....	645	625
July 7	....	....	....	....	....	....	....	....	....	....	649	625
July 20–21	....	....	827	807	....	....	742	705	....	....	....	....
August 5	....	....	....	....	....	....	....	....	....	....	704	....
September 8–9	....	....	819	801	....	901	734	709	....	....	....	....
October 5	....	....	....	....	....	....	....	....	707	658	667	621
October 28–29	....	....	....	....	....	....	....	....	717	663	709	625
November 16–17	....	....	809†	801	....	....	726	708	....	....	....	....
December 2–3	....	....	....	....	....	....	....	....	713	663	687	615
1938:												
January 4	....	....	....	....	....	....	....	....	....	....	684	616
January 7	....	....	....	....	....	....	....	....	712	666	....	....
January 11–12	....	....	811	808	....	892	729	708	....	....	....	....
February 2	....	....	....	....	....	....	....	....	....	....	681	613
February 10–11	955*	948	....	....	....	....	....	....	710	666	....	....
February 24–25	....	....	812	810	....	892	725	715	....	....	....	....
March 1–2	....	....	....	....	....	....	....	....	709	668	679	615
March 4	....	....	810	810	....	....	....	....	....	....	....	....
March 9–12	....	....	848	813	902	896	754	709	....	....	731	614

* At head of navigation (about 110 miles above Boulder Dam) on this date there was considerable silt at bottom (Elevation 1 068 ft).

† November 16: No silt layer in Las Vegas Wash area of lake; bottom elevation, 936 ft; no silt layer in Callville Wash area of lake; bottom elevation, 1 026 ft.

These results indicate fluctuation at the upper stations in the level of the top of the silt which probably represents flowing of silt toward the dam. The observations made at Pierces Ferry in February, and at the other stations in February and March, seem to indicate such a flow of turbid water. The notes submitted with the description of the sampling program for February included the following:

"As the head of navigation was approached there was a distinct line between the clear and turbid water. There were stretches in this vicinity constantly bubbling as if the air entrapped in the rocks on the bottom of the lake was still being released. Where these bubbles came up they carried with them a cloud of muddy water to the surface.

"When the heavy rains starting around March 4 began to bring up the flow of the Virgin and Colorado Rivers, an attempt was made to pick up its effect in the lake by sampling on March 5, 9, 11, and 12. In the sampling of March 9, at Virgin Canyon there was noticed an upstream surface current. At 259′ depth below the surface there was no noticeable deflection of the sampler cable but at 275′ depth below the surface the sampler cable showed a very decided downstream inclination as if encountering a strong downstream current."

Observations made at Boulder Dam and at Cape Horn, about 3 200 ft above the dam, show considerable fluctuation in the level of the top of the silty water. The data for Boulder Dam indicate that it may be difficult to observe, accurately, the elevation of the bottom of the lake, because the material deposited on the bottom appears at times to be consolidated, and, at other times, to be so stirred up or loosened that the sounding weight penetrates it to a greater depth. During the periods of observation the top of the silty water near the dam has been below the top of the old diversion dam, which is still (1938) in place.

The samples collected in Virgin Canyon show that the water in contact with the silt was more concentrated in September than in July. The quality of the water in the September samples from the bottom of the lake in Virgin Canyon was similar to that in a turbid flow observed at Grand Canyon a few days prior to the sampling in Lake Mead. These comparative data would seem to indicate that there had been a density flow through the lake, from the head of the lake, as far as Virgin Canyon, at least, and that the time required for the flow over that distance was about six days. Samples collected in Boulder Canyon in September indicate a possible movement of turbid water to that point, but the evidence for Boulder Canyon is not quite so satisfactory as that for Virgin Canyon. In addition to the samples taken in the deeper parts of the lake, samples of water have been taken at the surface in several parts of the lake and, for some sampling periods, at several depths in order to obtain information on the dissolved constituents at different parts of the lake and at different depths.

The stratification, or its opposite, the thorough mixing of the water in a reservoir, and the season of the year when such mixing occurs, may affect greatly the chemical quality of the water, whether wasted or drawn from the reservoir for use, and may determine the level from which it might be desirable to draw the water. In the summer of 1937, the surface water in Virgin Canyon, Boulder Canyon, in the lake between these canyons, and in the wide part of the lake at the mouth of Las Vegas Wash had practically the same concentration and was similar in composition to the water which flowed past the Grand Canyon station during the spring flood. At considerable depths below the surface during the same period the water had a concentration of 700 to 800 ppm of dissolved matter, which is practically the same as that of the water released from the reservoir during the fourteen months ending February 28, 1937. The presence of such a large body of water of uniform chemical composition seems to indicate that at some time, and by some means, the water in the lake became well mixed. During the summer of 1937 this large body of mixed water had a temperature of about 53°F and the uniformity of this temperature in different parts of the lake may be another indication of the thoroughness of the mixing. Temperature measurements made by the Bureau

of Reclamation show that in January, 1937, the water at all depths at the dam had a temperature of about 55° F. During the summer the water at the surface of the reservoir was warmer, of course, and, in July, midday temperatures of the water at the surface were as high as 90° F.

The depth-temperature curves in the vicinity of the dam show temperatures decreasing from the surface to a depth of about 200 ft where about 52° to 54° F were observed during the summer months. Below about 200 ft the same temperature was observed as far down as the top of the layer of turbid water where there was a sharp increase in temperature. It is not known how great an effect the mixing caused by a temperature "turn-over" may have on density currents and stratification; but the evidence seems to indicate that in the early winter of 1936–37 the entire mass of water in the reservoir came to a temperature that was essentially uniform, and it seems likely that the water throughout practically the entire reservoir was of essentially uniform chemical composition. It is not known whether the turbid water (if there was any at that time) lying at the bottom of the lake was involved in the "turn-over" of the winter of 1936–37; but it seems probable that its density may have been so great that little or none of it was so involved. The water brought into the lake by the spring floods carried considerable quantities of suspended matter, but usually those floods carry larger particles, chiefly, which settle quickly. The spring run-off was low in dissolved matter and its temperature was somewhat higher than that of the water previously stored in the reservoir. It seems likely, therefore, that the incoming flood water with its lower density, because of lower content of dissolved matter and higher temperature, spread out over the surface of the lake and mixed only slightly with the main body of stored water.

Observations made by the Bureau of Reclamation during January, 1938, at different points in the lake showed that temperatures ranging from 56° to 59° F, at the surface and in the top 100 ft of the lake, were quite uniform, and that at greater depths they varied between 53° and 56° F. In March, the temperature of the water at most depths was between 52° and 55° F. It seems likely that the top layers of water (possibly the upper 100 or 150 ft, at least) were mixed during a "turn-over" in the winter of 1937–38, but the available data are not sufficient to show that the "turn-over" affected greater depths. Samples of water collected at different depths show variations in the quantity of dissolved matter which would indicate that the mixing of the water was not thorough. The values of specific conductance given in Table 11 indicate the degree of concentration of the water at different depths.

The stratification throughout the lake is apparent from these data. The average values given are representative of values having a small range. At all sampling points the more highly concentrated water is found near the bottom of the lake, but the total quantity of such concentrated water is a small fraction of the entire body of water in the lake.

In a region where there is sufficient range in temperature to cause a "turn-over," the yearly cycle of a storage reservoir of large capacity that carries over considerable depths of water from year to year appears to the writers to consist of at least two phases: (1) The annual "turn-over," which so thoroughly mixes the water near the surface that it becomes essentially uniform in chemical

qualities and in temperature, and which does not involve, perhaps, a stratum at the bottom of the lake (where the water with its load of silt may be of such density that it does not mix with the remainder of the water at the time of the "turn-over"); and (2) the inflowing flood water that contains less dissolved

TABLE 11.—AVERAGE VALUES OF SPECIFIC CONDUCTANCE OF WATER AT DIFFERENT DEPTHS AT FOUR POINTS IN LAKE MEAD

(Specific Conductance,* $K \times 10^5$)

Approximate elevation (feet above sea level)	BLACK CANYON				BOULDER CANYON				VIRGIN RIVER NARROWS				VIRGIN CANYON			
	1937		1938		1937		1938		1937	1938			1937		1938	
	Oct.	Dec.	Jan.	Mar.	Sept.	Nov.	Jan.	Mar.	Sept.	Jan.	Feb.	Mar.	Sept.	Nov.	Jan.	Mar.
1 000 to 1 100	84	88	92	104	71	81	96	103	76	98	103	103	85	110	109	110
850 to 950	115	114	116	121	117	116	121	121	119†	99†	128†	184†	121	133	135	127
800	121	120	119	134	119	127	124	133	....	....	....	....	162†	138†	141†	155†
700	145	124	129	143	114†	123†	167†	166†	....	....	....	....	....	....	....	....
625	156†	132†	147†	153†	....	....	....	....	....	....	....	....	....	....	....	....

* Specific conductance × 8 = approximate parts per million of dissolved solids.
† Samples from the bottom of the lake.

matter and is less dense than the water already in the lake and, therefore, spreads over and rests upon it. During the summer, also, temperature differences develop which lead to more or less definite stratification which persists until the next "turn-over," when the cycle starts again. Into this cycle that is relatively uniform in the times and types of changes, there come the non-uniform and erratic turbid discharges carrying such fine silt and of such relatively great density that they flow as fairly definite streams along the bottom of the reservoir under the lighter clear water.

The apparent mixing of the lake water at times, and its failure to mix at other times, is of interest in relation to density currents which may not involve flows of turbid water but, instead, may involve flows of water of such varying concentrations of dissolved salts that they may be of concern to the users of the water discharged from the lake, as Mr. Hill has shown.

Any serious study of the problem of stratified flow will require the closest co-operation between laboratory and field. Data obtained from observations made under field conditions furnish the only sure knowledge of such complex phenomena. Because of the complexity, field measurements must be directed by laboratory experiments and theories in order to obtain necessary pertinent facts without an undue waste of time and effort.

Theories and criteria of stratified flow have been developed in laboratories, and values of the criteria, seemingly independent of the size of the surrounding apparatus, have been obtained from laboratory experiments. One such theory was described by Mr. Monish who made an attempt to apply it to the data presented in the paper. The application involved several assumptions as to density and velocity distribution, which are not immediately susceptible to

proof or disproof and which could be verified only by their determination under field conditions, including measurements of the magnitude and direction of currents and of the densities and viscosities of the waters in the reservoir so as to chart and define, completely, the strata under consideration. In order to expedite the task of obtaining such data and the necessary correlation as to places of measurements, it would be necessary to devise a combined current meter, direction indicator, and distant-indicating hydrometer—a difficult but not impossible task. A sonic depth-finder might be an aid in determining deposition of silt in a large and deep reservoir.

Assuming that these measurements have been made and have confirmed the criterion of stability found by the laboratory, the next step might be an attempt to control the currents.

Further study should include a large amount of field work to ascertain the position and extent of the stratum of turbid water and of the strata of water of different densities throughout lakes of various sizes. In a reservoir as large as Lake Mead, observations must be made in wide and narrow parts of the lake as well as systematically at the lower and upper ends. The continued study of the characteristics of the incoming water will be of fundamental importance in the interpretation of the results obtained from the studies within the lake. It would seem desirable to have observations made in reservoirs which would be typical of classes so as to have information on the effect of climate, topography, reservoir characteristics, and inflow characteristics on the occurrence of the phenomenon of density currents.

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DISCUSSIONS

ESSENTIAL CONSIDERATIONS IN THE STABILIZATION OF SOIL

Discussion

By C. A. Hogentogler, Assoc. M. Am. Soc. C. E., and E. A. Willis, Esq.

C. A. Hogentogler,[13] Assoc. M. Am. Soc. C. E., and E. A. Willis,[14] Esq. (by letter).[14a]—There are two essential parts to an examination of soil as a material for use in highway construction: (1) The field survey, and (2) the laboratory analysis.

Field surveys are made to establish the environment of the soils in place or in the locations in which they are to be used. Such surveys include studies of climate, topography, underground drainage conditions, and service behavior of existing roads under prevalent conditions. Both the American Association of State Highway Officials and the American Society for Testing Materials[15] have adopted methods for surveying and sampling sub-grade soils. The surveying and sampling of soils are supplemented by laboratory tests. The type of tests and the extent of testing required depend on the purpose for which the soil materials are to be used.

In the paper, emphasis was placed on the character and properties of the soil materials as revealed by laboratory tests. It was assumed that satisfactory field conditions would be accomplished.

Highway engineers have been accustomed to select aggregates almost exclusively on the basis of their hardness, durability, and grading. These aggregates are mixed with Portland cement and bituminous binders with the expectancy of equal serviceability, this being possible because the designs utilize

NOTE.—The paper by C. A. Hogentogler, Assoc. M. Am. Soc. C. E., and E. A. Willis, Esq., was published in June, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1937, by Jacob Feld, M. Am. Soc. C. E.; and April, 1938, by Donald M. Burmister, Assoc. M. Am. Soc. C. E.

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[14] Assoc. Highway Engr., U. S. Bureau of Public Roads, Washington, D. C.

[14a] Received by the Secretary April 1, 1938.

[15] A. A. S. H. O. Method T-86, Adopted, 1934; Standard Specifications for Highway Materials and Methods of Sampling and Testing, A. A. S. H. O., 1935, pp. 213–221; also, A. S. T. M. Designation: D 420–35T, pp. 940–949, *Proceedings*, 38th Annual Meeting, Vol. 35, 1935.

average strengths of groups of materials instead of the maximum strengths of the best mixtures individually; but in soil stabilization engineers strive for the maximum strengths of individual mixtures of soil aggregate and binder. They study films of air, moisture, chemical solutions, and insoluble binders; their relative adhesion to soil particles; and how their strengths and thicknesses are influenced by traffic, climate, and admixtures. Both Mr. Feld and Professor Burmister have indicated the desirability of a generally accepted system of soil classification for engineering purposes. Professor Burmister has described in his excellent discussion the multiplicity of factors which must be considered in any such grouping.

The authors are convinced that the practical method of attack is to group soils on the basis of performance for each particular engineering use. Simple tests may then be used to identify the individual members of each performance group.

Four groupings of soil materials based on performance when used for a particular purpose in highway construction have been adopted. They are: (1) A grouping for sub-grade materials; (2) a grouping for embankment materials; (3) a grouping for graded materials for use in base or surface courses; and (4) a grouping for fine-grained soils stabilized with Portland cement or bituminous material. On the basis of the test data, the soil samples are classified according to these groups and studied further in an effort to improve on the test result limits tentatively established.

The general grouping for sub-grade soil materials in which eight groups are recognized is widely used by State highway departments. The tests used for the identification of these materials are: (*a*) Mechanical analysis; (*b*) liquid limit; (*c*) plastie limit; (*d*) shrinkage limit; (*e*) field moisture equivalent; and (*f*) in some cases only, the centrifuge moisture equivalent.

A grouping for embankment materials has been adopted as a standard specification for the American Association of State Highway Officials.[16] The specification includes six groups for embankments less than 10 ft in height, and five groups for embankments greater than 10 ft in height. The tests used for identification in these groupings are: Liquid limit; plastic limit; and maximum weight, per cubic foot, as determined by the compaction tests.

The grouping for graded materials for surface and base course construction has been adopted as a standard for the American Association of State Highway Officials.[17] Seven groups—three for surfacing materials and four for base course materials—are included. These materials are identified by the liquid limit and plastic limit tests, and the mechanical analysis.

The grouping for fine-grained soils stabilized with Portland cement and bituminous materials has been submitted to the American Association of State Highway Officials in part. Two groups of materials, differentiated in the case of stabilization with Portland cement by wetting and drying, and freezing and thawing tests, are recognized. The compaction test is used to determine the quantity of water to add during construction.

[16] Standard Specification M 57–38, A. A. S. H. O.

[17] Standard Specifications M 61–38 and M 56–38, A. A. S. H. O.

Results of check tests conducted during the nine years, 1929–1938, by the Bureau of Public Roads in co-operation with the various State highway testing laboratories show that tests for identifying the members of the various soil groups have been developed to an entirely satisfactory status.

The widespread usage of these groupings has indicated that they are of practical value and that the employment of specifications based on them are workable and produce results which are more satisfactory than those previously attained.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PRACTICAL APPLICATION OF SOIL MECHANICS
A SYMPOSIUM

Discussion

BY WILLIAM L. WELLS, JUN. AM. SOC. C. E.

WILLIAM L. WELLS,[75] JUN. AM. SOC. C. E. (by letter).[75a]—Mr. Buchanan is to be congratulated on his presentation of the Mississippi River levee problem, and for his suggestions for improving the design of future developments by the application of a knowledge of soil behavior. Although, as a whole, the present sections have satisfactorily passed the test of actual usage, it is believed that better designs, for future extensions, can be effected by use of the rational methods offered by soil mechanics. The writer wishes to discuss briefly his observations of levees at various localities and certain studies which have been made by him. Each design feature will be discussed separately.

Side Slopes.—The method of designing the side slopes is based on determination of soil constants and computation of the stabilities of various slopes of a given height by means of the so-called "method of slices." The position of the center of the critical failure surface must be determined by trial and error. A tedious graphical construction is required for each trial. The method advanced by D. W. Taylor, Assoc. M. Am. Soc. C. E., in 1937,[76] is a tremendous improvement over the graphical procedure. Mr. Taylor has determined the position of the center of the critical failure surface for different slopes and values of ϕ, the angle of internal friction of the soil in question. He has obtained a mathematical solution for the stability analysis and has shown his results in the form of nests of curves, by means of which a stability problem

NOTE.—This Symposium was presented at the meeting of the Soils Mechanics and Foundations Division, at Boston, Mass., October 7, 1937, and published in September, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: September, 1937, by the members of the Committee of the Society on Earths and Foundations; November, 1937, by Messrs. S. C. Hollister, T. T. Knappen, and L. F. Harza; December, 1937, by Edward Adams Richardson, Esq.; January, 1938, by Messrs. Richards M. Strohl, William P. Creager, Jacob Feld, and Y. L. Chang; February, 1938, by Messrs. Charles Senour, Donald M. Burmister, and Donald W. Taylor; and April, 1938, by Messrs. Lee H. Johnson, Jr., and Gregory P. Tschebotareff.

[75] Asst. in Chg., Soils Section, U. S. Engr. Office, Memphis, Tenn.

[75a] Received by the Secretary March 24, 1938.

[76] "Stability of Earth Slopes," by Donald W. Taylor, *Journal*, Boston Soc. of Civ. Engrs., Vol. XXIV, No. 3, July 3, 1937.

can be solved in a few minutes after values of soil constants have been determined.

Mr. Buchanan mentions briefly the possible effects of forces, created by seepage, upon the stability of the side slopes. A few studies made by the writer indicate that consideration of these forces is of the utmost importance. This is borne out by the results of Mr. Taylor's studies. These forces are created by the loss of potential in the direction of flow. The force acting upon a soil mass of a given volume is proportional to the hydraulic gradient of flow through that mass and acts in the direction of flow. Fig. 70 illustrates the general

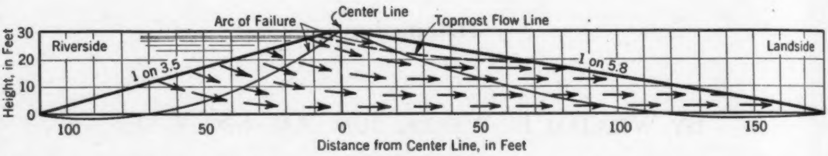

FIG. 70.—DIRECTION OF SEEPAGE FORCES THROUGHOUT CROSS-SECTION, STEADY SEEPAGE CASE

directions of seepage forces throughout the cross-section of a levee assuming that flow through the section is established during the high-water period.

As can be noted, the effect of these forces is to increase the stability of the river-side slope and decrease the stability of the land-side slope. This condition of seepage, through the land-side slope, has been designated as the "steady seepage case" by Mr. Taylor. His studies and those made by the writer indicate that, for the land-side slope, this is the most dangerous condition that can occur. Consequently, the land-side slope should be designed so as to be stable when subject to this condition of seepage.

Fig. 71 illustrates the general directions of seepage forces throughout the cross-section immediately after the passage of a flood and consequent drawdown of impounded water.

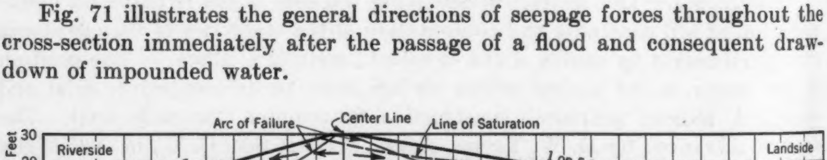

FIG. 71.—DIRECTION OF SEEPAGE FORCES THROUGHOUT CROSS-SECTION, SUDDEN DRAW-DOWN CASE

The effect of seepage forces in this case is to decrease the stability of both the river-side and land-side slopes, the former being affected to a greater extent than the latter. Mr. Taylor has designated this condition of seepage, through the river-side slope, as the "sudden drawdown case." The studies,[76] previously mentioned, indicate that, for the river-side slope, this is the most dangerous condition that can occur. Thus, the river-side slope should be designed so as to be stable when subject to this condition of seepage. These studies also indicate that the "sudden drawdown case" is more dangerous than the "steady

seepage case" when the draw-down period is of less duration than the period required for development of flow through the section. This condition generally exists in the Mississippi Valley. If homogeneous material is assumed, it follows that the river-side slope should be of less inclination than the land-side slope in order to obtain the same degree of stability for both slopes. The river-side slope most commonly used for levees in the Lower Mississippi Valley is one having an inclination of 1 on 3.5 (*B* section). Experience has shown that, in the great majority of cases, this slope is stable when subject to flood conditions. Thus, it appears that, in so far as stability is concerned, a land-side slope having an inclination somewhat greater than 1 on 3.5 would be adequate.

Control of Seepage.—As Mr. Buchanan states, seepage through the levees and its control have formed the bases for the design of the standard levee sections. Land-side slopes have been flattened in order to prevent out-cropping of seepage on this slope. It is interesting to note that in the case of pervious levees on relatively impervious foundations, model and other studies[77] indicate that a reduction in the inclination of the land-side slope results in a reduction in the inclination of the slope of the topmost flow line; however, the amount of seepage through the structure is decreased. For instance, assuming a levee 30 ft high, with a 10-ft crown, a river-side slope of 1 on 3.5 and a 1-ft free-board, the height of the point of emergence of the topmost flow line above the base, in the case of a land-side slope of 1 on 3.5, is 16.5 ft, whereas for a land-side slope of 1 on 5.8 (*B* section), it is 17.4 ft. The seepage passing through the structure is reduced about 35% by the reduction in slope. In the case of levees founded on relatively pervious foundations, the position of the topmost flow line appears to depend largely upon under-drainage conditions. If drainage conditions are such that a change in the inclination of the land-side slope would affect the position of the topmost flow line, it is probable that a reduction in the inclination of the land-side slope would cause a reduction in the inclination of the topmost flow line.

A reduction in the inclination of the land-side slope causes an increase in the average length of the path of percolation; hence, there is an increase in the time required for the development of flow. Observations made during the 1937 flood in the Memphis (Tenn.) Engineer District showed, in general, no apparent seepage through the levees, except in the case of sand levees; hence, it is evident that the average length of the path of percolation and permeability of material for the *B* section are such that flow does not develop during a normal flood period. It is not surprising that, in some cases in the past, when steeper land-side slopes were used, a dangerous volume of seepage cropped out on the land-side slope. This is probably due to the fact that no selection was exercised over available materials, and, as a result, layers or lenses of sand extending from river side to land side would exist in some cases. At present, no selection is exercised over available materials; however, reduction in the inclination of the land-side slope has decreased the probability of the occurrence of continuous sand lenses.

For future extensions, it is believed that this purpose could be more economically accomplished by the use of steeper land-side slopes combined with under-

[77] "Hydraulic-Fill Dams," by Glennon Gilboy, Assoc. M. Am. Soc. C. E., First Cong. of Large Dams, Vol. 4, Rept. No. 46, pp. 231–267.

drainage of these slopes largely accomplished through selection and proper placement of available materials. Should seepage develop through these sections during flood periods, its amount would be very small and it would not adversely affect the safety of the structures.

In the case of sand levees, the method used by the Vicksburg (Miss.) District for controlling excessive seepage seems to be excellent.

Under-Seepage.—Uncontrolled under-seepage, and resultant "sand boils," are by far the major causes of trouble along the levee system during flood periods. The fundamental cause of "sand boils" has been ably discussed[78] by L. F. Harza, M. Am. Soc. C. E. He demonstrates that "sand boils," or flotation, can occur only when the hydraulic gradient at the point of escape of under-seepage on the land side equals $(1 - P)(S - 1)$, in which P = the percentage of voids of the material, and S = the absolute specific gravity of the material. For all sands, the value of this expression is about 1.0.

The B section has a base width of $10\ H$ in which H is the maximum design head against the structure. The base width is the shortest path for under-seepage and the average hydraulic gradient along this path is, therefore, $\frac{H}{10\ H}$, or 0.1. Under these circumstances, it would appear that "sand boils" could not occur unless foundation conditions, or the plan of construction were such as to create high escape gradients on the land side of the levee. A few typical "sand boil" areas have been inspected by the writer and, in general, the following has been found: Subsoil conditions consist of a shallow (5 ft to 10 ft) depth of relatively impervious top-soil under-laid by a deep stratum of sand. River-side borrow-pits have been excavated into this impervious top-soil, thus exposing the sand stratum directly to water. The impervious blanket is continuous on the land side. Under these conditions, the quantity of under-seepage is limited by the quantity that can pass up through the impervious land-side blanket and, as a result, the loss in head between the river-side pit and the bottom of the impervious land-side blanket is negligible. Then, the major part of the head on the structure is dissipated in upward flow through the land-side blanket. This would tend to lift and disrupt the blanket. A "sand boil" would occur at any weak point in the blanket, that is, through a crack, fissure, or hole due to these conditions and concentration of outflow at the weak point. The presence of high hydro-static pressures beneath the land-side blanket, as indicated by flowing wells, was noted by the writer during the 1937 flood in typical "sand boil" areas.

For future extensions, where these foundation conditions prevail, it is believed that these difficulties can be avoided by leaving the impervious blanket intact on the river side and allowing drainage of the under-lying sand on the land side by locating borrow-pits there. Under these conditions, practically all the head against the structure would be lost in flow downward through the river-side blanket, and, consequently, there would be no tendency for "sand boils" to develop on the land side. This procedure might increase the volume of under-seepage. However, the quantity would be small provided the

[78] "Uplift and Seepage Under Dams on Sand," by L. F. Harza, *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1352.

river-side blanket were continuous for a reasonable distance (500 ft to 1 000 ft) away from the levee. Assuming a 30-ft head, an underlying stratum of coarse sand ($K = 500 \times 10^{-4}$ cm per sec) of 200 ft thickness, and a length of blanket of 800 ft, the volume of seepage would be about 0.0083 cu ft per sec per ft of levee. Since these unfavorable foundation conditions would prevail for only relatively short distances, the drainage system behind the levees would probably be more than adequate for the disposal of seepage.

The prevalent method of strengthening such weak places in the existing system is placement of a blanket over the affected area on the land side. At present, experiments with river-side blankets and seals are planned.

Stability of Foundations.—This phase of earth dam or levee design has been completely covered by Mr. Hough. Under the heading, "Laboratory Testing," he states that he made no effort to determine an apparent angle of internal friction for clay samples. The writer believes that no effort should be made to determine this factor for any type of material since the value obtained (quick shear tests) is meaningless in practically all cases and usually not on the safe side in the case of non-saturated silts when tested according to more or less standard procedure (vertical loads of 0.2, 0.4, and 0.6 ton per sq ft). This seems to be due to the fact that, in a given time of test, the percentage consolidation increases with an increase in intensity of vertical loading. Thus, the apparent angle of internal friction is often considerably greater than the true angle and serious error would result if extrapolated values are taken from the shear curve. Shear strength of material in place should either be determined from the results of quick shear tests performed at the least vertical loads practicable, or from the results of delayed or consolidated shear tests combined with a determination of pre-consolidation pressures.

Mr. Hough's model and analytical studies are most interesting. Fig. 41 summarizes the three well-known methods of determining shearing stress in the foundations of earth dams. Jurgenson's formula for the "rigid boundary" case cannot be expected to yield exact results since, in addition to the assumptions mentioned by Mr. Hough, vertical loading is assumed. The fallacy of this assumption has been discussed[79] by Glennon Gilboy, Assoc. M. Am. Soc. C. E. According to this formula (Equation (9)), the shearing stress induced in a given foundation by dams having the same side slopes but different heights, is constant. If the reaction, or total earth pressure, between the two halves of the dam is considered as the cause of shearing stresses, it can be demonstrated that the average shearing stress along the base of a dam, for the foregoing case, is proportional to the height of the dam. In computing earth pressures, if a hydrostatic pressure ratio, K, of 0.5 is assumed, it can be demonstrated that these stresses exceed Jurgenson's stresses when h is greater than 4 a. Thus, in the case of high dams on relatively shallow plastic foundations, Jurgenson's method, the photo-elastic method, or the Haines method would indicate small stress intensities, whereas actual stress intensities might be of considerable magnitude. In the same manner, it can be demonstrated that, for the "infinite depth" case, average shearing stresses along the base of the structure can exceed Jurgenson's values when L is less than h.

[79] *Engineering News-Record*, February 10, 1938.

Since no assumptions are made in the case of the photo-elastic method, the results obtained by this method should be correct. Thus, it appears that the data from which the curve of Fig. 41 was plotted must have been obtained between the aforementioned limits.

None of these methods considers stresses induced in the foundation by water pressure acting on the reservoir side of the dam, nor the important stresses created by seepage or drainage forces in partly consolidated foundations. The effect of the former would probably be to increase shearing stresses in the land-side foundation. The effect of the latter would depend largely upon direction of drainage; if horizontal, stresses would certainly be increased, possibly to a value, $p \times \frac{2\,a}{L}$ (assuming no consolidation at the end of a construction period, and a small ratio, $\frac{a}{L}$). The effect of these forces could be determined approximately by use of the Haines' method combined with flow nets.

The action of these forces explains the peculiar behavior of certain Mississippi River levees. These levees are impervious structures built on impervious soft foundations, generally overlaid by a 5-ft to 10-ft stiff clay blanket. In one case, heaving occurred 850 ft from the center line of the levee at a weak place in the overlying blanket. This behavior strongly suggests a fluid-like transmission of pressure. For future construction in such areas the placement of a drainage blanket over a part of the base seems to offer definite possibilities of relieving such conditions.

Valuable data concerning the behavior of soft foundations might be obtained by a continuation of Mr. Hough's studies. The factors discussed herein could be considered and their effects determined.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

THE DESIGN OF ROCK-FILL DAMS

Discussion

BY MESSRS. JOHN E. FIELD, JOHN H. WILSON, FREDERICK H. FOWLER, I. C. STEELE AND WALTER DREYER, AND F. KNAPP

JOHN E. FIELD,[21] M. AM. SOC. C. E. (by letter).[21a]—The author is to be commended for bringing this newer and less known rock-fill dam to the attention of the profession, for the very practical suggestions he has made, and for suggesting dimensions based on his experience for criticism and further development and change. Although the paper is quite complete and very instructive, there are experiences among individual engineers which, if presented, would be illuminating. With the growing appreciation of the value of drainage in dams of all types, the rock-fill dam should find a wider and more general acceptance.

Assuming proper design in all cases the writer lists the three types of dam in point of economy of construction, thus: (1) The earth-fill; (2) the rock-fill; and (3) the concrete dam. All must be 100% safe because less than absolute safety cannot be tolerated.

Some designers assume too hastily that, since dumped rock assumes a natural slope, this natural phenomenon offers a perfect guide in design. This assumption may, or may not, be correct, depending on what is meant by a "natural" slope; whether it is the slope on which the rock will stop moving when dumped, or the slope on which, when at rest, the rock will start moving. Observed natural slopes are: For the lee side of sand dunes, 1 on 1.5, and sliding will start when a small quantity of sand is excavated from the foot of the slope; in rock slides composed of small flat and shell-like rock, the angle is generally less than 30°, or about 1 on 1.8; and in rock slides of large boulders the observed slope is often as much as 40°, or about 1 on 1.4. These slides were made by

NOTE.—The paper by J. D. Galloway, M. Am. Soc. C. E., was published in October, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1937, by Messrs. Cecil E. Pearce, and H. B. Muckleston; January, 1938, by Harold K. Fox, M. Am. Soc. C. E.; February, 1938, by Messrs. Charles H. Paul, and A. Floris; March, 1938, by Messrs. Howard F. Peckworth, Oren Reed, Walter L. Huber, Samuel B. Morris, and L. F. Harza; and April, 1938, by Messrs. Paul Baumann, O. W. Peterson, and George W. Howson.

[21] Cons. Engr., Denver, Colo.

[21a] Received by the Secretary March 25, 1938.

rock falling from cliffs which, by reason of their inertia, would develop flat slopes; also, the action of frost and the elements over long periods of time have flattened them. In excavating a bench across such rock slides or old mine dumps, or cutting through them, the side slopes of the cut are generally made much steeper than the surface slope although at present the tendency is to flatten the slope to prevent rock, loosened by frost or rain, from rolling into the cut. Too many stable and successful dams have been built with slopes less than 1 on 1.3 (see Table 1) to justify the adoption of that slope as a minimum. Although the writer does not disagree with the suggested practical 1 on 1.3 slope for rock-fills he feels that steeper slopes are preferable unless structural conditions and operations require the flatter slope. The steepest slope permissible is that at which movement would start but, as far as the writer knows, this has not been determined by research, or by experiment with different materials, placed by different methods.

The difficulty encountered in connecting the impervious facing to the abutments suggests making a temporary connection of some flexible material, such as lumber, which would last well beyond the period of settlement.

In a rock-fill dam, with all the uncertainties of a reliable, permanent, impervious facing, one must assume that water may pass into the fill and the presence of any erodible material should be avoided.

Two of the dams listed in Table 1 have failed. The Castlewood Dam, in Colorado, was washed out in 1933 by over-topping. The spillway was in the center of the dam, about 200 ft long and 4 ft below the crest at either end; the lower slope was in steps built up of large rectangular rock and then grouted. Many smaller floods had passed through the spillway and over the stepped lower slope, but in the 43 yr the grouting, never first class, had largely disintegrated, the flood of 1933 had probably loosened and dislodged some of the steps, and the material in the body of the dam was rapidly washed away. This material was composed of rock, spalls, and sand; probably the entire material from the quarry, with some of the over-burden, was used. On three occasions prior to 1904, also, piping had washed out the foundation material. After the third wash-out an earth blanket on a slope of 1 on 3 was placed up stream from the dam and was effective in preventing further trouble. The design or dimensions of this rock-fill dam were not at fault, but poor foundations and poor workmanship were to blame for the three wash-outs and the final collapse.

The Beaver Park Dam, in Colorado, has been an economic failure because water in large volume—130 cu ft per sec—passed beneath and around the dam, through the abutments. Apparently, the canyon in which the dam was built had no rock in place, either at the bottom or on the side slopes. It appears that a slide had occurred from the high cliff to the east and choked the ancient river channel, and water passing through this broken up mass of rock had eroded a new channel. After the serious leaks appeared, the bottom and east slope were drilled and grouted but, due to lack of funds, were not extended sufficiently into the old slide and the leak at present is greater than the normal flow of the stream. It has been suggested that an earth blanket be placed in the area above the dam both against it and on the hill slope to a point up

stream and beyond the area of the old slide. The crest of the dam appears to have moved about 3 ft down stream and to have settled nearly as much; the rock used was rather soft and friable. In this case, a lack of proper borings and ignorance of the geological conditions are quite apparent.

Fig. 18 shows the section of a dam in which the rock is graded; such grading will probably decrease the percentage of settlement. The plan differs from those of most rock-fill dams in that the Class *A* is large rock and that Class *B* next up stream is from 3 in. to 15 in., and the next is 3 in. and smaller, with a filtering medium of sand (Class *D*) between Class *C* and the earth-fill. Classes *B*, *C*, and *D* are put through a crusher and screened. If the fill is made in horizontal lifts, it is apparent that the line of contact between the several classes of rock can be altered—made steeper or flatter as desired. The lower slope (1 on 2) seems flatter than necessary, and being flatter than the natural dump slope, probably requires some extra work in placing (see Fig. 18). Suggested steeper slopes are shown by the broken lines, *A B* and *A C*.

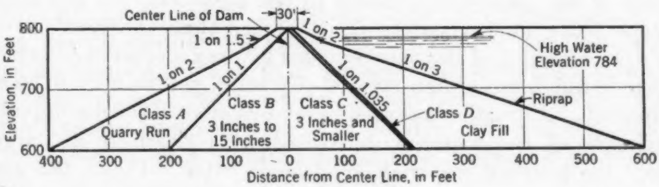

FIG. 18

Core-walls in the center of a dam are inadvisable not only on account of inaccessibility but more because the full hydrostatic pressure is at the center of the structure, leaving to that part above the core-wall the sole duty of supporting the rip-rap and the core-wall. With settlement of the structure and the raising and lowering of the water in the reservoir, changing stresses are imposed on the core, and, if it is an earth-fill, the part above the core becomes saturated, with a decided tendency to slide into the reservoir. Resultant stresses, first on one side and then on the other, tend to rupture the core-wall. A core composed of screened gravel, however, would be of benefit in lowering the line of saturation, and the rock-fill dam can be considered an extreme case in which the "core" occupies almost the entire section of the dam.

Where, for any reason, it is desirable to use all the materials from the quarries, including the generally condemned "fines," one can visualize a core of screened rock obviating any danger of lack of proper drainage in the fill.

Under some conditions gates and collapsible stop-planks can be used in spillways; for example, if the discharge capacity has been determined for a 200-yr period on a 100-yr record automatically collapsible stop-planks could be installed to meet the 1 000-yr probability. Gates placed in a spillway are generally undesirable and to rely on the watchman or attendant at the dam is dangerous; if, however, there is a power-house or factory at the dam where numerous and competent men are always at hand, gates in spillways could be tolerated.

JOHN H. WILSON,[22] ASSOC. M. AM. SOC. C. E. (by letter).[22a]—Observation of the sandstone in a dam near Oneonta, Ala., will be of interest to those who are contemplating the use of sandstone for rock-fill purposes. By definition Mr. Galloway excludes this earth and rock-fill type; but the character of the rock and the quarrying methods are those that would be encountered at this site regardless of the type of dam.

The rock-fill design contemplated the placing of four gradings of stone. The construction plant was set up to meet the original specification of (quoting):

"348,072 Cu. Yds. Class (A) shall be run of quarry. It is expected that 50% will be larger than one-man stone.
"314,584 Cu. Yds. Class (B) shall be crushed stone of from 3 inches to 15 inches in size.
"336,552 Cu. Yds. Class (C) shall be crushed stone ½ inch to 3 inches in size.
"36,916 Cu. Yds. Class (D) shall be screenings ½ inch or less in size."

The site of the quarry is adjacent to the dam area, the elevation of the quarry floor being approximately 50 ft higher than the crest elevation of the dam. Over-burden of clay was removed by drag scrapers and carried down to a sandstone which was of decomposed character, yet too hard to excavate by such equipment. A dragline was used to remove more of the decomposed stone. Test specimens of the material uncovered were classified as "argillaceous sandstone," the bonding material being ferric oxide. Under this stone is a harder, more highly consolidated material.

Primary blasting was conducted by use of well-drill holes spaced about 15 ft on centers and loaded with an average charge of 950 lb of dynamite. The detonating agent used was cordeau.

The resulting fragmented rock was loaded by 4-yd steam shovels and hauled in 12-yd side-dump tractor wagons to a 48-in. by 60-in. jaw crusher. From the primary crusher the stone went on to a grizzly that removed the material smaller than 8 in. to a vibrating screen which made the final classification of the material. A secondary 20-in. gyratory crusher received a part of the stone that passed over the grizzly, and fed the fine material to the same grizzly and vibrating screen for reclassification.

Stone too large to be manipulated by the 4-yd shovels is reduced in size by continuous secondary drilling and blasting. The operations described have resulted in the introduction of a large percentage of fines smaller than 0.5 in. The primary blasting and secondary blasting have produced a large quantity of these fines, and the crushers have added to the total. Between 15% and 20% of the total yardage produced will screen smaller than 0.5 in. The quantity of rock broken down by blasting, to sand or fines, is so excessive that it has not been possible to load the larger stone to the exclusion of this fine material. The loads of quarry-run stone containing, by visual inspection, too great a proportion of fines have been wasted.

In order to accommodate the use of the fines produced at the screen, the thickness of the up-stream blanket of Class D rock was increased from 5 ft to

[22] Engr., Walsh Constr. Co., Indianapolis, Ind.

[22a] Received by the Secretary March 25, 1938.

8 ft. A further effort was made to include the fine material in the dam by introducing thin horizontal blankets over the Class B and Class C rock.

Mr. Galloway has requested that discussion be directed to problems of design. The writer's purpose in relating the methods used in the construction of this sandstone rock-fill dam is to direct the attention of designing engineers to conditions that may exist in another sandstone formation similar in character to that described. In the writer's opinion sluicing of fines into the larger rocks would produce a more dense fill than has resulted in the past. This might give greater stability to the structure, but it also might prove objectionable from the standpoint of open drainage for water which seeps past the facing.

The exclusion of fines from a quarry operation in rock of this nature will increase the cost of construction. Not only by reason of rehandling, but during rainy weather the fines form a gritty mud which clogs screening equipment, resulting in serious retardation of construction operations.

The writer would suggest alternate methods of writing specifications covering construction that involves the placing of sandstone fills:

(1) Eliminate all attempt to classify the sandstone by crushing and screening, placing the run-of-quarry stone in the fill. Sluice the fine material to prevent any accumulation of fines in a local area. In order to provide a section that would be stable, both the up-stream and down-stream faces of the rock-fill would probably have slopes of 1 on 2, or flatter. The resulting increase in quantity would not necessarily mean an increase in price or cost due to the lowered unit cost of producing the fill material.

(2) If it is anticipated that a part of the quarried stone may be unsuitable for use, it seems to the writer that the basis of payment for rock-fill should be yardage in cut; or yardage in fill, plus an allowance for yardage placed in a designated waste dump. This would permit the construction engineer more latitude in his selection of suitable stone for the fill. This kind of clause in the specification should be reflected in the price bid for the work by permitting the bidder to reduce his allowance for coverage of ambiguous contract provisions.

In considering the design of a rock-fill dam with rock of questionable character, due regard should be taken of the construction problems which will influence the cost of construction and thereby affect the economy of design.

FREDERICK H. FOWLER,[23] M. AM. SOC. C. E. (by letter).[23a]—The definition of a "rock-fill dam" adopted by the author, well describes the type of structure that has reached its maximum development in the Sierra Nevada of California. It does not, however, include the many important variants of the rock-fill type that have developed elsewhere, particularly in regions where foundation conditions are more difficult and common labor cheaper than in the California mountain districts.

Even in the rock-fill structures of the Sierra there has been a rapid evolution and, in the later dams, such as Bucks Creek and Salt Springs, in California, the placed-rock sections are far thinner in relation to the greater heights, and conform more closely to the author's definition than did the earlier Relief Dam, in

[23] Cons. Civ. Engr., San Francisco, Calif.

[23a] Received by the Secretary March 28, 1938.

California. At Relief, for example, with a maximum height of only 140 ft, the combined horizontal thickness of the facing and placed-rock section at the base of the dam was 100 ft. At Salt Springs, with a maximum height of 328 ft, the adopted combined horizontal thickness of the facing and placed-rock was only 25 ft. At Relief, in other words, the base thickness of the placed-fill section was more than 70% of the total height of the dam; at Salt Springs, it was less than 8 per cent.

These designs thus tended toward a simplicity of structure, an increasing proportion of drop-fill, and a corresponding decrease in rubble fill and mortar-placed facing, an evolution that may be considered as typical of practice in the Sierra Nevada.

It will be noted, however, that no design reached the ultimate limit of the drop-fill type, namely, a complete drop-fill structure as in the Cogoti Dam, in Chile,[24] with no derrick-placed cushion. Nor have the designers of dams in the Sierra of California been driven by difficult foundation conditions, or tempted by a plentiful supply of cheap labor, to increase the proportion of derrick-laid rock or to abandon drop-fill entirely.

Drop-fill, however, has been definitely abandoned in Algeria.[25] The most recent technique in hand-placing and derrick-placing of fill and the design of facing, etc., for four large Algerian dams, is set forth in an article[26] published since the author's paper. This article gives (in convenient form and in English translation) material hitherto scattered in various publications in several foreign languages.

From a western viewpoint, the Algerian designs are notable for the care and thoroughness used in the construction of the entire main fill of derrick-placed rock of the largest sizes practicable, with the crevices well packed with smaller rock and quarry spalls. Settlement in the body of the dam is reduced to a minimum by care in placing the fill, and further reduction and equalization of settlement, due to poor foundations, are sought by the construction of thick rubble foundation slabs, on which the rock-fill rests. Under these slabs are very complete systems of under-drains. Heavy cut-off walls, provided with inspection and work galleries, run the full length of the up-stream base of the dams, and deep grout curtains at the up-stream sides of the cut-offs were used as an additional precaution against seepage. Highly specialized laminated facings, varying in detail at the several dams, are characteristic of all four structures. Although none of these dams conforms to the author's very specific definition of a rock-fill dam, they can scarcely be placed under any other general classification.

An even more specialized rock-fill structure is the Shing Mun Dam,[27] 275 ft in height, designed to supplement the water supply of Hong Kong and Kowloon. According to the description, the dam consists of five parts. Its face is a rela-

[24] "Earthquake-Proof Dam in Chile," *Engineering News-Record,* November 5, 1931, p. 725.

[25] "Note sur le Masque en Béton Armé du Barrage de Bakhadda," by M. Drouhin, *Annales des Ponts et Chaussées,* 1935, VIII, pp. 281–291.

[26] "Algerian Dams of Placed Rockfill," by I. Gutmann, *Engineering News-Record,* December 2, 1937, p. 889.

[27] "Steep Concrete Face on Rock-fill Dam in China," *Engineering News-Record,* November 12, 1936, p. 677; also, Second Congress on Large Dams, Washington, D. C., September, 1936.

tively thin diaphragm built of rich reinforced concrete and divided, by vertical bitumen-filled joints, into segments, so that it can stand considerable distortion without destruction; a cut-off wall is formed by an extension of the face downward to impervious rock and built of rich unreinforced concrete placed for maximum density. Back of the face of the dam and the cut-off wall is a "thrust-block" of low-grade mass concrete, resting on the surface rock and designed to transmit the water pressure to the rock-fill behind. Finally, in order to insure a uniform transmission of pressure between the thrust-block and the rock-fill, a wedge of dry sand is inserted between these elements. The rock-fill was hand picked and brought up in 2-ft layers sloping toward the thrust block. Drains, and inspection pits and galleries, are provided. The Shing Mun design is "a far cry" from the simple designs outlined by the author, and is probably on the border line between what may, by broadest definition, be called a rock-fill, and a composite structure of uncertain classification. It demonstrates to a marked degree the changes in a basic idea that may be brought about by vastly different costs of labor and material, in combination with more difficult foundation conditions.

Through the migratory tendency that is common to engineering designs, Algerian practice in placing fill and foundation treatment has invaded Mexico, and Chilean practice in laminated concrete facing has traveled to California.

The use of derrick-placed and hand-placed fill in the Mexican project (the Tepuxtepec Dam, on Rio Lerma, Michoacan) probably resulted from the employment of an Italian consultant, thoroughly familiar with North African practice. Its use was made economically possible by the large available supply of cheap labor. Derrick-placed fill, rubble-masonry foundation slab, drainage, and deep grout curtain follow, closely, Algerian practice, under generally similar conditions. The design of the backing for the up-stream facing, however, departs from Algerian and other practice, since it consists of a series of horizontal blocks (with inclined up-stream edges) extending back varying depths into the fill, and tending to tie the fill and facing support together. This unusual block design is merely another example of the many attempts to secure a proper adjustment of the impervious facing to the progressive settlement of the rock-fill body of the dam—one of the main problems in the successful design of this general type of structure.

What is termed the "Chilean type" of laminated concrete facing is that used on the Cogoti Dam in that country. In the article[28] describing its details, it was stated that the same type of facing was to be used on seventeen major rock-fill dams then proposed by the Chilean Bureau of Reclamation; however, according to the most reliable information now available, construction of all these dams, except the Cogoti Dam, has been postponed on account of adverse conditions in world finance, and no details are available concerning the results secured through its use on the Cogoti Dam. That structure was to be built entirely of loose fill, with slopes conforming to the angle of repose of the material used in the structure. A facing of concrete was to be poured against a gravel blanket on the up-stream slope and the laminated facing of reinforced concrete laid directly on this base slab. This plan to secure a stable and water-

[28] *Engineering News-Record,* November 5, 1931, p. 725.

tight structure without using any expensive derrick-laid rubble is naturally very attractive, if practicable, and it is to be hoped that a full description of the measure of success attained at Cogoti may be forthcoming.

The results attained in Chile are of particular interest, since a very similar type of laminated facing was used initially on San Gabriel No. 2 Dam, in California, and was extremely unsatisfactory, with the settlement that developed in that structure. Different methods of construction might have reduced the settlement in the fill, and with a reduced settlement the facing might have held. An answer to the question as to whether the failure of the facing was due to excessive settlement or to an inherent weakness in the laminar type of facing may be had when the results in Chile are known.

Selection of construction methods that will produce a fill (of either placed or dropped rock) with a relatively small and generally uniform settlement, and a type of facing of sufficient flexibility to follow such settlement in the fill as cannot be avoided, constitute two of the main problems in design. Under Algerian conditions, with relatively poor foundations and high value of water, difficulties of maintaining a tight face with large settlement in drop-fill, have led to the definite abandonment of that type of placement.

Even where drop-fill is to be used there remains either much to be learned concerning the best construction methods, or a very considerable adjustment of divergent engineering views as to what is accomplished by certain methods already generally accepted. All authorities appear agreed, for instance, that plentiful sluicing of the fill during construction will facilitate and accelerate settlement, thus making it possible to construct the final facing at an earlier date and with less fear of harmful distortion than would otherwise be possible. However, aside from the general agreement that lubrication of the larger pieces is one of the most beneficial results of sluicing, there are divergent views concerning its purpose. One description states that the purpose of sluicing is "to wash the fines into the fill," and another states with equal conviction that the purpose is "to wash the fines out of the fill." Possibly more detailed descriptions of local conditions and construction methods in these particular projects would reconcile these apparently discordant statements—in the meantime, the reader is somewhat at sea.

No more valuable contribution could be made to the discussion of rock-fill dam design than a thorough analysis of the settlement that has taken place, both during construction and after completion, in various dams now actually in operation. This is particularly true of large and unusual structures such as Dix, in Kentucky, Salt Springs, Bucks Creek, San Gabriel No. 2, and Cogoti Dams. Measurements are already available in brief form for Bakhadda Dam,[29] of the Algerian derrick-placed type; these measurements show not only the settlement that took place during the initial filling of the reservoir, but also the elastic recovery that occurred upon emptying the reservoir.

It would also be interesting to know the settlement that has been recorded, and the action of the welded-steel facing, in the El Vado (gravel-fill) Dam, which, so far as the writer knows, is one of the largest steel-faced fill dams now (1938) actually completed.

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[29] *Engineering News-Record*, December 2, 1937, p. 892, Fig. 5.

The difficulties that have arisen in designing concrete or other permanent facing that will conform without rupture to the considerable initial settlements of very large rock-fill dams now commonly constructed, warrant favorable consideration of timber facing; this, although of relatively short life, will last well through the early period when settlement in the rock-fill is largest and most rapid. By the time that it becomes necessary to renew the timber facing most of the settlement will have taken place, and far greater freedom may be used in the selection of facing design.

I. C. STEELE[30] AND WALTER DREYER,[31] MEMBERS, AM. SOC. C. E. (by letter).[31a]—The following discussion of Mr. Galloway's excellent paper will be principally confined to the subject of the design and settlement of Salt Springs Dam, in California, the principal design features of which are described by Mr. Galloway. Reference is also made to a paper[32] by O. W. Peterson, M. Am. Soc. C. E., which includes a description of the construction methods used in building the dam, and to an article,[33] "High Rock-Fill Dam with Jointed Concrete Face," by I. C. Steele, which includes a plan of the dam, a table showing slab thicknesses and reinforcing, and a table of elevations and offsets used in laying out the vertical and horizontal curvatures of the up-stream face. Fig. 19, a view of the up-stream face shortly before completion of the rubble facing, shows various features and elevations pertinent to this discussion and to the aforementioned references.

A brief summary of the controlling features of design and construction should be sufficient for the purpose of this discussion.

Factors Influencing Selection of Rock-Fill Type of Dam.—Several factors influenced the decision to build a rock-fill type of dam at Salt Springs. The dam site offered a choice between the rock-fill and concrete gravity types. Preliminary estimates indicated a saving of about $2 000 000, including indirect and overhead charges, for the rock-fill structure. The intended height, 328 ft above general foundation level, did not greatly exceed the height of the successful Dix Dam in Kentucky. Unlimited quantities of good quality rock were readily available for quarrying at desired elevations. The construction of a large capacity spillway, separate from the dam, was feasible. Studies of available data, and experience with other dams, indicated that a rock-fill dam of the desired height could be built and operated with the utmost safety.

Consideration Given to Settlement in the Design Assumption.—Settlement at any point on the face was estimated to be the vector sum of the vertical settlement due to the weight of rock in a vertical column of rock extending from face to foundation and the movement due to water pressure acting on an inclined column of rock normal to the face and having a length equal to the distance between the face and the foundation. An arbitrary value was given to the coefficient of settlement or compaction for this purpose.

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[31] Asst. Chf., Div. of Civ. Eng., Dept. of Eng., Pacific Gas & Electric Co., San Francisco, Calif.

[31a] Received by the Secretary April 1, 1938.

[32] *Proceedings*, Am. Soc. C. E., August 1930, p. 1319.

[33] *Engineering News-Record*, Vol. 104, p. 92.

Design of Face.—Details of face design are shown in Fig. 20 which also shows methods subsequently adopted for the repair of cracks and joints. A face concave in vertical section was selected to insure compressive forces acting toward the fill in the event that settlement differed greatly from the design assumption. A convex face might be pushed outward especially if there was an abrupt change in direction. Consideration was given to several methods

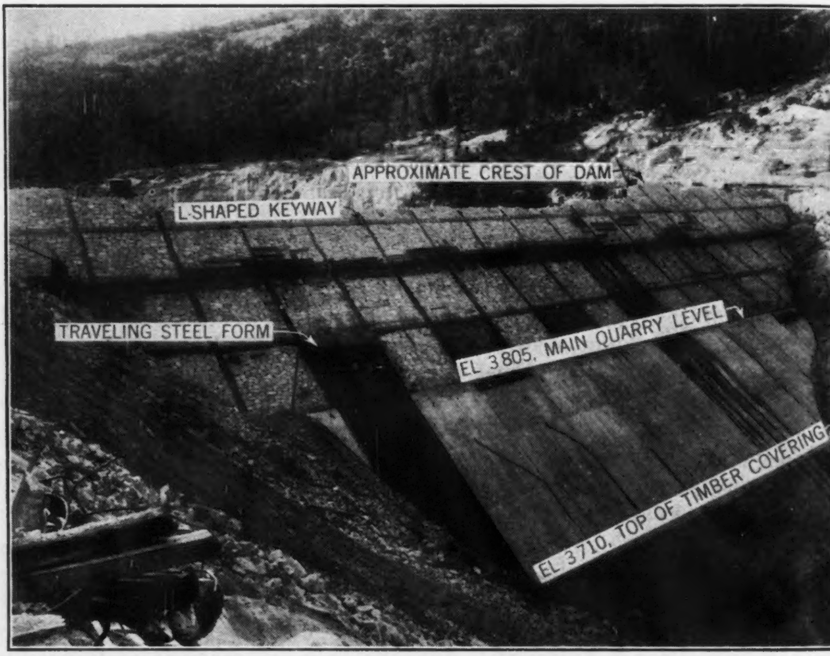


FIG. 19.—UP-STREAM FACE DURING CONSTRUCTION, SALT SPRINGS DAM

of designing the concrete face for the dam. The several designs considered and the reason for eliminating some and selecting the one are:

(1) Sliding Type Face.—This design consisted of a single thickness of slab supported on a rubble face which has been made smooth with concrete and on which asphalt has been applied. The concrete face would be separated from the rubble and, theoretically, would be free to move independently from the body of the dam. This plan was discarded after some study because it was believed that with water pressure acting on the dam, the face could not move as it was intended to, because construction inequalities and settlement during construction would make the supporting face quite irregular. There also was objection to the relatively slim column to withstand compressive forces developed by movement of the dam, particularly when water pressure was not acting on the face.

(2) Laminated Face.—This type was never given serious consideration because of the desire to make the face quite substantial. A laminated face

cannot be anchored successfully to horizontal keyways unless the keyways, in turn, are supported on rigid struts extending upward from the cut-off in the plane of the face.

(3) A Monolithic Slab.—A slab of substantial thickness, and with the concrete worked well back into the interstices of the first layer of dry rubble, was built so the face would have sufficient rigidity to withstand compressive forces resulting from settlement of the rock-fill on which the face was laid. During the early stages of construction the interstices in the exposed surface of the first layer of rubble were left open. This method was later changed and all large openings, and many of the smaller ones, were chinked in order to prevent undue waste of concrete and to lessen the possibility of movement of fresh concrete from the slab proper into voids in the rock section as the traveling steel forms were moved ahead.

Joints in Face.—Consideration was given to a continuously reinforced concrete face, but one consisting of panels about 60 ft square, with joints along the periphery, was selected partly because of anticipated movement of the fill from each bank toward the center of the canyon in a direction parallel to the axis of the dam, which movement would induce very heavy compressive stresses in the face, and also because studies showed that if the dam settled in the manner and to the degree which was estimated, the stresses would be so great that the face would fail structurally. The adopted type is essentially a number of panels approximately 60 ft square with copper water-stops in the construction joints. Horizontal joints (see Fig. 20) are designed without an elastic filler, so the downward compressive forces can be transmitted directly from one panel to another. Inclined joints are made with a 1-in. thickness of elastic material to permit some movement parallel to the face. Other dams have placed two joints at each inclined rib, but one was considered sufficient for this dam. The copper seal was designed to have sufficient strength to withstand a separation with a maximum head of 4 in. without exceeding usual working stresses. Horizontal keyways were made triangular in shape to avoid overhanging rock during construction. They also served as walkways and work platforms during this period.

Settlement of Face.—The only data available with respect to settlement of rock-fill dams was for the settlement of the crest. The settlement obtained under the design assumptions described in the foregoing indicated that the greatest movement that would be likely to occur after the face was poured was in the lower elevations of the dam. Therefore, reference points were set in the concrete at a corresponding corner of each panel and the movement of these reference points was measured at frequent intervals during the first three years after setting and once each season thereafter.

Fig. 21, with Table 2, shows selected settlements at each measuring point for indicated periods. The extent of vertical, horizontal, and diagonal movement of any point may be obtained by scaling. Initial readings are plotted along straight lines at elevations which are averages of the two end points in each row. Joint opening or closing, as measured along the line of points in Row *H* and caused by lateral movement of the rock-fill, is shown at the top.

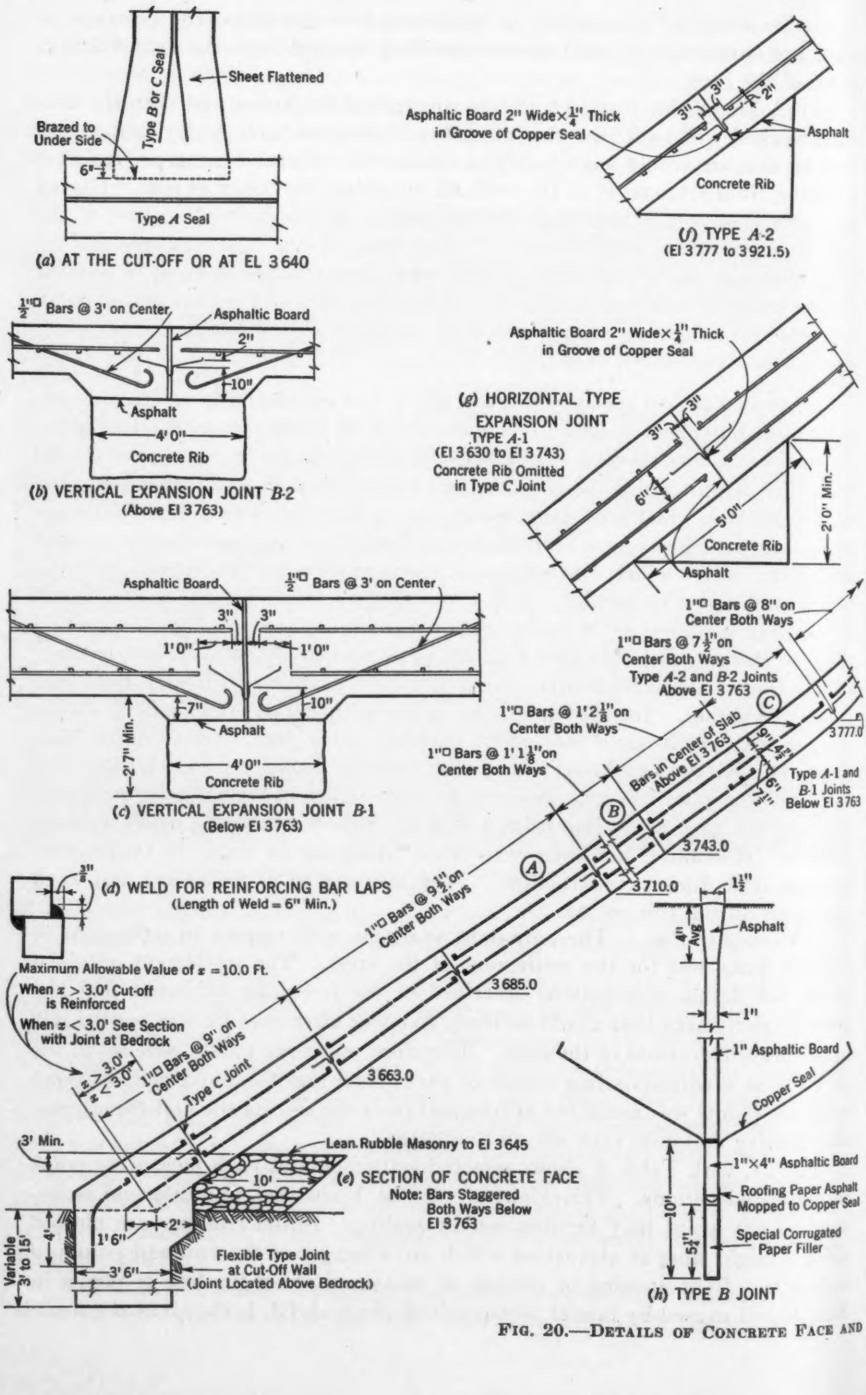
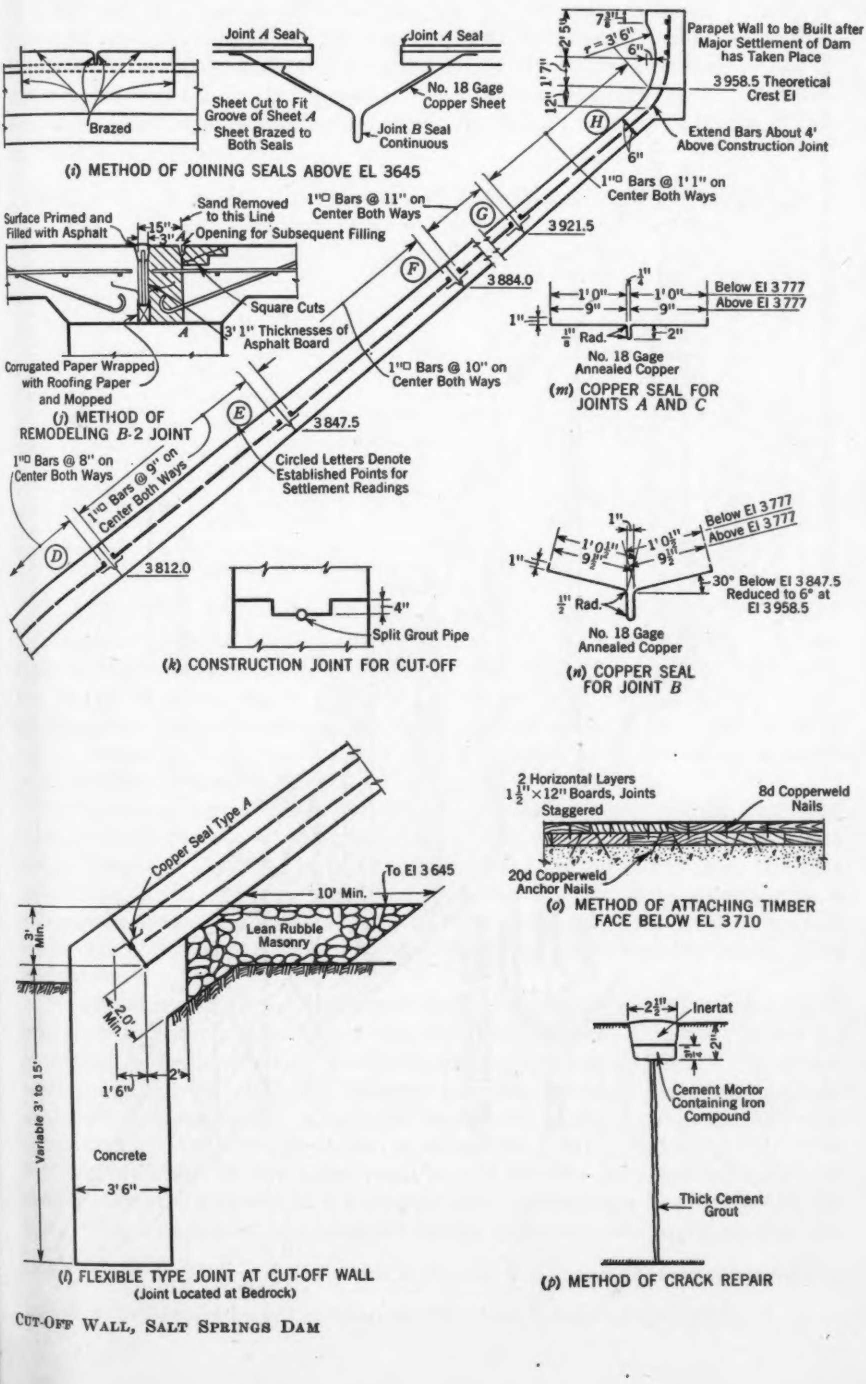


FIG. 20.—DETAILS OF CONCRETE FACE AND



CUT-OFF WALL, SALT SPRINGS DAM

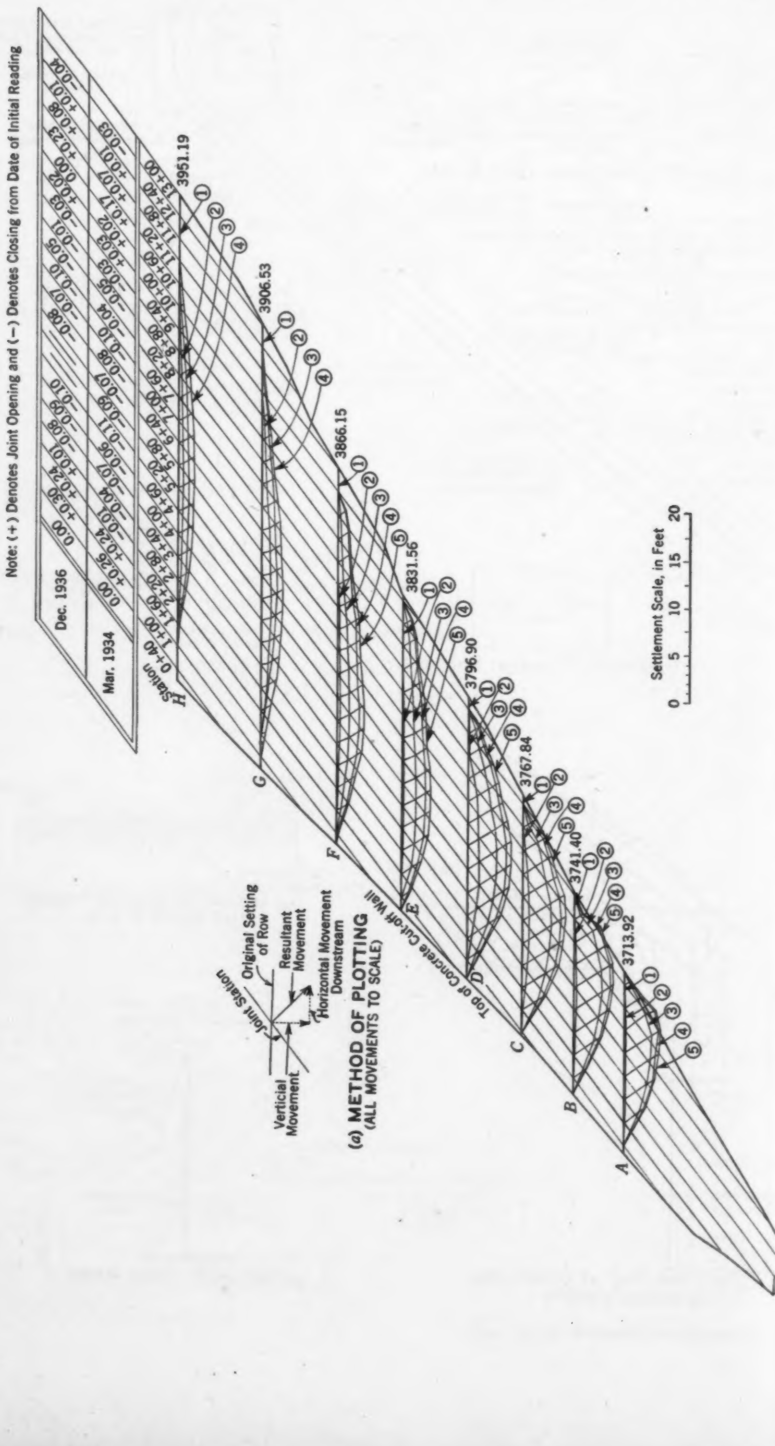


FIG. 21.—DIAGRAM SHOWING SETTLEMENT OF UP-STREAM FACE OF SALT SPRINGS DAM

The maximum movement in a direction normal to the face was 5.41 ft at Point *D*, Station 4+60. Of this amount, 1.41 ft occurred prior to the first filling of the reservoir, and 4.0 ft occurred during and subsequent thereto. The latter value approximately represents the normal settlement caused by water pressure. Settlement almost equal to the foregoing occurred at Points *C* and *D*, Stations 5+20 and 5+80.

TABLE 2.—NUMBER AND DATE OF OBSERVATIONS PLOTTED IN FIG. 21

Row	Original Setting		Before First Filling		After First Filling*		After Second Filling		After Third to Fifth Fillings	
	Observation No.	Date	Observation No.	Date	Observation No.	Date	Observation No.	Date	Observation No.	Date
H	1	May 9, 1931	..	....	2	February 15, 1932	3	September 1, 1932	4	December 1, 1936
G	1	March 10, 1931	..	....	2	February 17, 1932	3	September 1, 1932	4	December 3, 1936
F	1	February 10, 1931	2	April 9, 1931	3	February 18, 1932	4	October 4, 1932	5	December 3, 1936
E	1	February 10, 1931	2	April 24, 1931	3	February 17, 1932	4	November 1, 1932	5	December 3, 1936
D	1	May 28, 1930	2	April 10, 1931	3	September 28, 1931	4	November 21, 1932	5	February 28, 1937
C	1	May 8, 1930	2	April 4, 1931	3	November 2, 1931	4	December 27, 1932	5	February 28, 1937
B	1	May 28, 1930	2	March 6, 1931	3	December 10, 1931	4	March 16, 1933	5	February 20, 1935
A	1	May 1, 1930	2	March 6, 1931	3	February 15, 1932	4	March 16, 1933	5	February 14, 1934

* Reservoir rose only to maximum elevation 3 910, in 1931.

Fig. 22 shows settlements at various measuring points along the face for four vertical sections parallel to the river. The vertical distances between spillway crest level and points of maximum measured settlements for the several sections of fill which exceed 150 ft in height average about 60% of the corresponding vertical distances from spill crest to foundation level. The point of actual maximum settlement in each vertical section no doubt occurs at some level between measuring points.

As mentioned previously, the settlement at any point on the face was assumed to be the vector sum of two components, one the vertical settlement due to weight of the fill in a vertical column of rock extending from the face to the foundation and the other movement due to water pressure acting on a column of rock extending from the face to the foundation in a direction normal to the face and having a length equal to the distance between the face and the foundation.

The greater part of the first component occurs while the fill is being made and prior to the construction of the concrete face and, therefore, is not an important factor in affecting the structural behavior of the slab. The settlement that occurred after the concrete face was installed has been carefully measured and analyzed. Maximum settlement of the crest under the first component and for a height of dam of about 330 ft has been nearly 2 ft. The vertical settlement at any other point on the face due to weight of rock only should vary as the square of the vertical distance from the face to the foundation. This component of settlement at any point on the face, therefore, can be expressed as $2.0\left(\frac{h}{H}\right)^2$, in which h = height of point above the foundation and H = maximum height corresponding to the settlement of 2.0 ft.

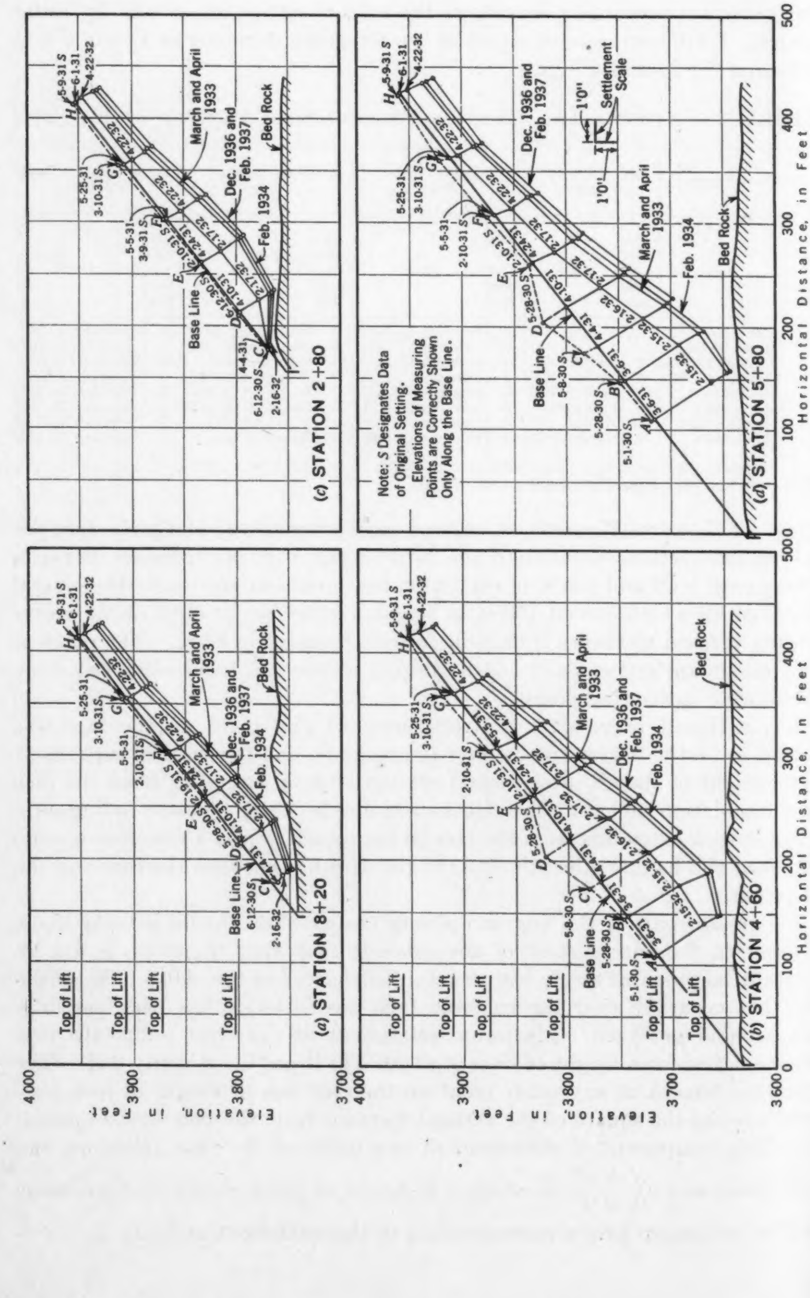


FIG. 22.—PROGRESSION OF SETTLEMENT OF SELECTED SECTIONS, SALT SPRINGS DAM

If the coefficient of settlement could be taken as a constant, the vertical displacement due to water pressure could be expressed by the factor,

$$S = \frac{P\,h}{K} \qquad (1)$$

in which P is the vertical component of water pressure and K is a "modulus of settlement" similar to Young's modulus for an elastic substance. Owing to storage operation it is not always possible to measure the settlement of all points on the face of the dam. At the time of the latest complete observation the settlement of the crest was 1.75 ft. If the vertical settlement due to the weight of the fill only at other points is deducted from the total settlement by using the formula, $1.75\left(\frac{h}{330}\right)^2$, the resulting amount would be due to water load only, and if the foregoing assumptions are valid, the factor, K, can be determined.

The result of this determination is shown graphically in Fig. 23, which has lines of equal resistance to vertical settlement under water load. Each line has the same value for the "modulus of settlement"; hence, the total settlement due to water load can be computed by Equation (1).

It will be noted that around the periphery of the canyon, the modulus has values of 300 000 to 400 000 lb, but is considerably higher at the center of the canyon above Elevation 3 800, which was approximately the main quarry level.

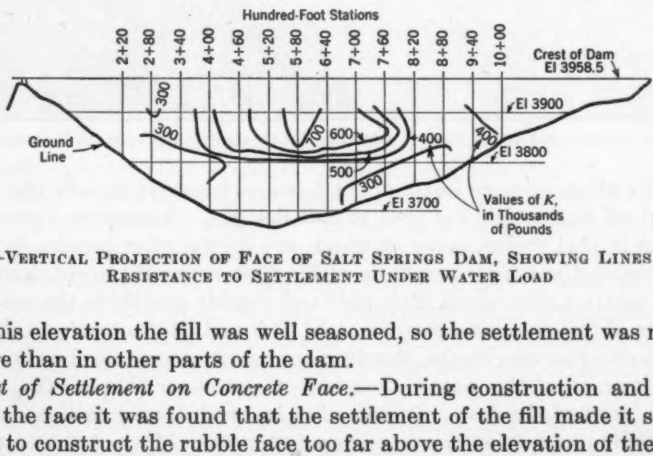


FIG. 23.—VERTICAL PROJECTION OF FACE OF SALT SPRINGS DAM, SHOWING LINES OF EQUAL RESISTANCE TO SETTLEMENT UNDER WATER LOAD

Below this elevation the fill was well seasoned, so the settlement was relatively less there than in other parts of the dam.

Effect of Settlement on Concrete Face.—During construction and prior to pouring the face it was found that the settlement of the fill made it somewhat of a risk to construct the rubble face too far above the elevation of the concrete face. Tremendous forces are transmitted from the loose fill to the more rigid rubble face and in a few cases massive granite stones 4 to 5 ft thick at the bottom cracked, or were slightly displaced, under these loads when the rubble wall was carried approximately 100 ft above the concrete face. It is best to let the fill be well seasoned before placing either the rubble or the concrete in order to reduce these stresses to a minimum. In one case these forces were so great that a concrete slab about 2.5 ft thick and with two layers of reinforcing steel, split in the plane of the upper layer over a triangular shaped area

50 ft long and having a maximum width of 17 ft. Similar action occurred over several other small areas in the immediate vicinity.

Settlement under water load demonstrated that greater flexibility was necessary adjacent to the canyon walls and in future work the design would

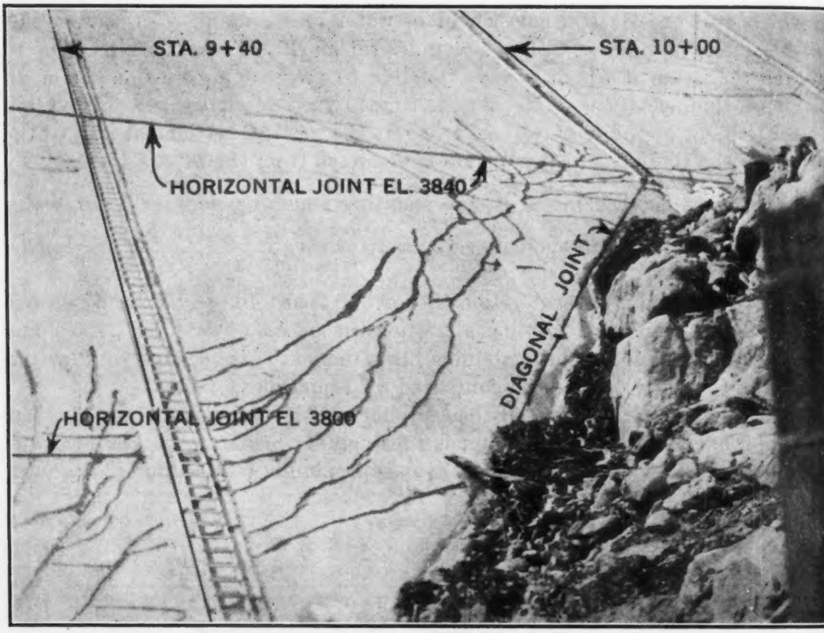


FIG. 24.—VIEW SHOWING CRACK REPAIRS IN CONCRETE FACE, SALT SPRINGS DAM

provide for three or more joints about 5 ft apart instead of only the one joint at the cut-off wall which was used at Salt Springs. A number of small cracks developed in this region, some of which are shown, after repairs, in Fig. 24. With few exceptions these cracks lie within 60 ft of the abutments and most of them lie within a zone about 20 ft wide and roughly parallel to the cut-off wall. The limits of this zone are approximately 10 ft and 30 ft away from the cut-off wall. As many as nine cracks, closely spaced, appear at some elevations. The uppermost crack of this nature does not extend closer than 60 ft, vertically, below the crest of the dam. Cracks vary in width from hair-line dimensions to a maximum of 0.5 in. in one or two instances. Most of the cracks were about $\frac{1}{16}$ in. wide. It has been necessary to chip out and fill these cracks, the method of repair being shown in Fig. 20. These repairs were made in 1932, 1933, and 1935, at a total cost of about $15 000, a nominal sum in view of the magnitude of the dam. At present writing (1938) leakage charts indicate that few additional cracks have developed since last repairs were made.

Movement of the fill toward the center of the canyon caused eight of the inclined joints to close completely, three to such an extent that the compressive stress caused spalling in the upper levels. It was necessary to reconstruct

partly three of these joints. The uppermost panel at Stations 3+40 and 4+60, and the two upper panels at Station 5+80 were repaired in the manner indicated in Fig. 20 at a cost of less than $4 000. In future work of this nature the space between adjacent panels will be increased to 2 in. or more in those joints that are likely to be compressed.

Thickness of Rubble Face.—Mr. Galloway mentioned two items given particular consideration when the thickness of the rubble face was being determined for the design, namely, that water pressure indicated that the thickness should be greatest at the bottom; and settlement under vertical load, being greatest at the top, indicated that the rubble should be thickest at the crest, so that a uniform thickness finally was adopted. In the light of the experience at Salt Springs during construction and since, the writers, who participated in the original discussion and agreed to the selection of the uniform thickness, now believe that the thickness near the bottom should be greater than at the crest and that the thickness at the crest could, in a well constructed dam, be less than was used at Salt Springs. The reason for this opinion is that most of the settlement of the crest occurs before the rubble wall and concrete face are constructed, so it is not necessary to have a heavy rubble wall at that level to protect the concrete face. The maximum total movement of the face occurs at about one-third the height of the dam and the greatest differential movement occurs at the bottom and around the periphery of the face near the canyon walls. The greatest protection, therefore, should be had at these critical points.

Furthermore, as brought out by the late D. C. Henny, M. Am. Soc. C. E., in his discussion[34] of Salt Springs Dam, the local sliding factors in the rock adjacent to the up-stream face are high in relation to the coefficient of friction of the rock. In the case of a dam like Salt Springs, the sliding factors at the bottom level are 77% at the face, reducing to about 60% at the back of a 25-ft thick rubble wall (thickness measured normal to the face). It appears desirable in this part of the dam to have a rubble wall with the rock well keyed together so that horizontal movement need not be resisted by frictional forces entirely. This prompts the writers, in the light of all the foregoing factors, to suggest, in the interest of consistent design of high rock-fill dams, a rubble face having a thickness normal to the face of 10 ft at the crest and increasing about 5 ft for each 100 ft of height. It may be advisable to go even further than this by arbitrarily increasing the thickness of the rubble face for some distance out from the canyon walls, as this area of the face is almost certain to develop some cracks.

Conclusions.—It is believed that the crest settlement at Salt Springs Dam is less in proportion to its height than the measured settlement of any other structure of similar cross-section. In the opinion of the writers, the principal reasons for this and for the successful performance of the structure throughout are:

(1) Adoption of essentially natural slopes of loose rock-fill for the up-stream and down-stream faces;

(2) Construction of the loose fill, with quarry-run rock of good quality granite;

[34] *Proceedings*, Am. Soc. C. E., August 1930, p. 1325.

(3) Placement of loose fill from high lifts (75 ft or more) resulting in a well compacted fill;

(4) Liberal sluicing of loose fill during placement operations;

(5) Delayed construction of a well-built dry rubble section to form a cushion of reasonable thickness between the impervious membrane and the loose fill; and,

(6) Delayed construction of a monolithic type reinforced concrete jointed impervious membrane of substantial proportions on the up-stream face of the dam.

F. KNAPP,[35] ESQ. (by letter).[35a]—The art of designing rock-fill dams, as developed during nearly a century of progress, and based on the experience gained by American engineers, is contained in this valuable paper. Abroad, notably in Algeria, several dams of the type discussed by the author have been built by French engineers, following, in general, the design principles stated in the paper. The innovation introduced consists in vibrating the rock-fill for the purpose of keeping down the voids and, therefore, diminishing the settlement of the structure. The Bakhada Dam, for example, with a height of 45 m (147.6 ft), an up-stream slope of 1 on 0.86 at the top, 1 on 1 at the bottom, and 1 on 1.25 for the down-stream slope, showed a horizontal deflection at the crown of 0.30 m (11.8 in.) and a vertical settlement of about the same magnitude, or 0.7% of the height. The rock-fill consisted of blocks with an average weight of 5 metric tons, resulting in 32% of voids. Increasing the size of the blocks to 10 tons reduced the voids to 26% as evidenced by the dams of Bon-Hanifia and Grib.

The Bakhadda Dam, designed originally with a double slab of reinforced concrete and only vertical joints, the lower one with a thickness of 30 cm (11.8 in.), suffered only the small settlements mentioned, with the result that the lower slab, with the reservoir filled, showed neither cracks nor displacements. The drainage amounted to 15 liters per sec (0.5 cu ft per sec) diminishing to 0.1 liter per sec after the second slab was built. It is interesting to note that the space below the joint of the upper slab is being drained by means of small channels.

The concrete slabs of rock-fill dams should be made as elastic as possible in order to be able to follow the settlements of the rubble cushion, and the speed of placing the rock-fill should be kept down to an economic limit for the purpose of allowing the fill to settle under its own weight. Vibrating the fill and sluicing contribute to a possible increase of the speed of construction.

[35] São Paulo, Brazil.

[35a] Received by the Secretary February 19, 1938.

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DISCUSSIONS

ECONOMICS OF THE OHIO RIVER IMPROVEMENT

Discussion

BY W. E. R. COVELL, M. AM. SOC. C. E.

W. E. R. COVELL,[31] M. AM. SOC. C. E. (by letter).[31a]—An analysis of the economics of the Ohio River improvement was presented by C. W. Kutz,[32] M. Am. Soc. C. E., in 1926, who covered the period from 1905 to 1925, inclusive. It is true that improvements have been made in the collection of commercial statistics, which afford more complete data with respect to the movements of river commerce from origin to destination. However, intimate knowledge of the sources and treatment of the data which were presented in the initial paper, particularly with respect to the upper end of the Ohio River and its principal commercial head-water, the Monongahela River, with their long history of heavy traffic, lead to the belief that the economic status of the Ohio River improvement to the end of 1925, as accepted from General Kutz's paper, is reasonably adequate. Colonel Hall's paper, therefore, makes available a complete although composite analysis of the economics of the Ohio River improvement up to and including the calendar year 1934.

The author's economic analysis is predicated upon five basic assumptions, which he lists and discusses in the section dealing with this feature. It is believed that all the assumptions as made are tenable.

Under Assumption (1) it is considered that the entire reduction of costs in transportation of freight is eventually passed on to the consumer. A particular function of the Ohio River improvement is that it broadens and intensifies competition with respect to industries, such as iron and steel, situated both on the water and inland, with a favorable effect upon the prices the public must pay for the products of such industries. The general public also benefits directly in many other ways; for example, many electric utilities are situated on the river

NOTE.—The paper by C. L. Hall, M. Am. Soc. C. E., was published in October, 1937, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1937, by Eugene L. Grant, Assoc. M. Am. Soc. C. E.; February, 1938, by Messrs. Fred Lavis, O. Slack Barrett, and Edmund L. Daley and Forrest E. Byrns; and March, 1938, by J. E. Willoughby and W. D. Faucette, Members, Am. Soc. C. E.

[31] Lt.-Col., Corps of Engrs., U. S. Army; Dist. Engr., U. S. Engr. Office, Pittsburgh, Pa.

[31a] Received by the Secretary February 21, 1938.

[32] "The Relation of the Ohio River and Its Tributaries to Transportation in the United States," by C. W. Kutz, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 1106–1122.

and receive their fuel supply *via* water. The cheaper cost of coal is reflected in cheaper rates to the consumers of electricity.

The river improvements have exerted a regulatory effect upon the rail-rate structure for coal in the Ohio River Basin by inhibiting a tendency toward higher prices. In this manner the savings in transportation effected by water are at least indirectly realized by the general public. General Kutz stated that his investigation indicated that, in general, the rail-rate structure for coal and steel movements in the Ohio River Basin did not discriminate between points having water competition and those without such competition. Nevertheless, certain exceptions were noted which are still applicable. Short-haul rail rates on coal in the region at the head of the Ohio River where coal movements on the rivers are very large, appear to be considerably less in the valleys where there is water competition than in the valleys without river improvements.

As indicated by the author in his discussion of Assumption (2), to present a rational analysis of the economics of the Ohio River improvement it is necessary to resort to rail rates as the basis for determining transportation savings by water. As long as the comparison is made completely from origin to destination (that is, not limited to line haul alone), the method serves the purpose adequately and, it is believed, equitably.

The following difficulties attending the river-rail comparison may be cited: For long hauls involving the Ohio River, the rail distance between origin and destination is usually shorter, as stated by General Kutz in his paper. Moreover, more than one rail line or system may afford the same service, a matter which has a bearing upon capital investment in railroad facilities underlying the service. Undoubtedly, the revenue from the relatively high short-haul rail rates goes in some part toward making up possible deficiencies in revenue from long-haul rail service. The statement is also often made that a railroad system has no concern over the fact that the rates for certain short hauls may be practically the same as for much longer hauls, which may place certain areas at a competitive disadvantage with others. The average rail revenue per ton for the system is what counts. However, resort to average rail rates is manifestly not equitable in the comparison of the cost of the competing transportation services for the derivation of the savings, since only the rail rates applicable represent actual "out-of-pocket" costs to the shipper for the rail service between origin and destination.

The transportation savings as determined do not include those resulting from temporary or emergency freight rates which, during several recent years, were appreciable. The Association of American Railroads has petitioned the Interstate Commerce Commission for a flat increase of 15% in rail freight rates throughout the United States. If this increase or any substantial part of it should finally be granted, as seems possible at the present time, due to the financial straits of several of the railroad systems, a very material increase in mills per ton-mile will apply to the savings for the future Ohio River commerce even if the cost of haul on the Ohio River were also to increase as much as 15 per cent.

In Assumptions (3) and (4) Colonel Hall deals with general ferry traffic and the capital cost of the Ohio River improvement, respectively. The Ohio River,

in the calendar year 1935, accounted for 1 160 639 tons of general ferry traffic; in 1936, such traffic mounted to 1 309 405 tons. Canalization of the Ohio River has permitted this traffic to be handled more safely and expediently. Although it is difficult to place a value on these benefits, they are positive and constitute a credit to the waterway improvement, which has not been included in the economic analysis. It is believed that there is no particular point in attempting to estimate and charge the waterway with the costs of the Federal Lighthouse Service and Steamboat Inspection Service on the Ohio River, since the costs are comparatively nominal.

Assumption (5) is as follows: "In comparing expenditures and benefits, no allowance has been made for taxes." This means that although the rail rates reflect taxes on the capital investment in railroad facilities, the Federal investment in the Ohio River improvement is not subjected to an equivalent charge in the economic analysis. There is some room for difference of opinion on this score. The basic question is whether or not the Federal Government would have been further ahead had the same funds gone into private investment on which taxes would accrue to the Government. It is believed that an investment of the Ohio River type does not necessarily result in a reduction of private investment of like amount. The improvement of the Ohio River has resulted through a subsidy on the part of the Federal Government to the Ohio River Valley which is rich in natural resources. The Federal Government, in its paternalistic attitude, has fostered the development of the Ohio Valley to such a degree that the river improvement has become a national asset. Every large city in the country is located on a navigable waterway. It is believed that the return in taxes resulting from the development of the Ohio River Valley made possible by the improvement of the river far outweighs any possible loss in taxes occasioned by the Federal investment in the improvement of the river.

There may be cited numerous indirect, practical benefits from the Ohio River improvement not mentioned by Colonel Hall, which are not susceptible to monetary valuation. The improved Ohio River has recreational features that are widely utilized. The navigation pools afford storage of water for industrial and, in some cases, domestic water supply. Pumping heads have been reduced. During periods of emergency, such as the World War, the water-transportation medium with its reserve capacity was extremely valuable to the country since the rail mediums were congested.

By his assumptions, Colonel Hall has indicated clearly the basis of the economic analysis. Whatever deficiencies obtain are of a minor nature only, and it is believed that they are more than offset by the intangible and indirect benefits.

Colonel Hall discusses the adequacy of the cost of transportation by water as adopted for the determination of the unit savings on the Ohio River. He states that the savings on the two all-important items of coal and steel are based upon line-haul costs of 2 and 3 mills per ton-mile for principal coal movements and about 3 mills per ton-mile for steel movements. An indication of the adequacy of these rates may be had from the following derivation of the cost of transportation for a coal haul, say, from Point Pleasant, W. Va., at the mouth of the Kanawha River, to the mouth of the Miami River below Cincinnati, Ohio.

Annual Cost of Coal-Towing Unit for Operation on the Ohio River (Tow = 10 Standard Barges, Each 26 × 175 × 11 Ft; and the Navigation Season Is a Full Year).—

Cost of Equipment:	
Towboat, 1 200 hp, Diesel type	$325 000
Reserve towboat 1 per 10 active, charge	32 500
Barges in transit, 10 @ $14 000	140 000
Barges, 20 additional, at terminals (ratio 2 to 1)	280 000
Total	$777 500
Fixed Charges, Annual:	
Interest @ 6% on $777 500	$ 46 650
Plant depreciation (20-yr life)	38 875
Insurance marine, 2½% average life value	9 720
Workmen's compensation insurance (2.28% of payroll)	775
Cargo and liability insurance	18 750
Taxes, 2% average life value	7 775
Total	$122 545
Maintenance and Annual Repairs:	
Towboat, 1.1 @ $9 000	$ 9 900
Barges, 30 @ $500	15 000
Total	$ 24 900
Ship Supplies (1.1 Towboats):	
Paint, rope, waste, lubricants, etc.	$ 4 875
Ice, laundry, linen, etc.	1 125
Total	$ 6 000

Payroll (First Watch):	
1 Master, 12 months @ $350	$ 4 200
1 Chief Engineer, 12 months @ $225	2 700
1 Striker Engineer, 12 months @ $125	1 500
1 Watchman, 12 months @ $140	1 680
2 Deck Hands, 12 months @ $110	2 640
Payroll (Second Watch):	
1 Pilot, 12 months @ $275	$ 3 300
1 Mate, 12 months @ $165	1 980
1 Second Engineer, 12 months @ $200	2 400
1 Striker Engineer, 12 months @ $125	1 500
2 Deck Hands, 12 months @ $110	2 640
1 Cook, 12 months @ $140	1 680
2 Chambermaids, 12 months @ $75	1 800
15 Sub-total	$28 020
Relief crews	6 000
Total	$34 020

Subsistence:

15 @ 80 cts per day for 365 days	$ 4 380
Relief crews	780
Total	$ 5 160

Fuel Oil:

1 200 hp @ 0.6 lb per hp-hr = 720 lb per hr,
or approximately 96 gal per hr

Assume 18 hr average use per day and 365 days per season:
96 × 18 × 365 = 630 720 gal per yr
630 720 gal @ 5 cts = $31 535

Summary of Cost:

Payroll	$ 34 020
Subsistence	5 160
Fuel oil	31 535
Maintenance and repairs	24 900
Ship supplies	6 000
Fixed charges	122 545
Payroll, subsistence, and fuel oil cost of reserve boat, 0.1 of $70 715 (arbitrarily added)	7 070
Total annual operating cost	$231 230
Overhead @ 5% (Fleet Superintendent, Assistants, etc.)	11 560
Total annual cost (rounded)	$242 790

Cost of Transportation of Coal on Ohio River from Mouth of Kanawha River to Mouth of Miami River (Tow = 10 Standard Coal Barges; 1 Ton = 2 000 Lb; and No Back-Haul Freight).—

Length of haul, in miles	226
Normal speed, in miles per hour	5
Number of locks in reach of river	12
Size of locks, in feet	110 by 600
Lockages required by tow per lock	1
Lockage time, average for system, hours per lock	0.5
Length of river reach assigned to operation, each lock, in miles	0.5
Loading of tow (down stream only), in tons	8 500
Round-Trip Time, in Hours:	
Assembling and landing time (2 × 2 hr)	4
Lockage time (24 locks at 30 min)	12
Pool time, $\frac{452 - 12}{5}$	88
Sub-total	104
Lost time, all causes, assumed at 35%	36
Total, round trip	140
Navigation season, gross, in hours	8 760
Number of round trips	63
Loading per round trip, in tons	8 500
Commerce per year, in tons	535 500
Haulage per year, in ton-miles	121 023 000
Annual cost, in dollars, per unit tow	242 790
Cost, in mills, per ton-mile	2

The following explanation is pertinent to the derivation of the cost of transportation of coal on the Ohio River as presented in the preceding data. The size of the tow is assumed to be 10 barges. However, the towboat is capable of handling a tow of greater size—up to 16 standard barges—which would require two lockings through each lock. It is assumed that all the traffic proceeds through all the twelve locks on the reach. However, during a considerable part of the year there is sufficient open-river stage to permit the traffic to proceed past the locks through the navigable passes of the dams with the wickets in a lowered position. An exception will be the new Gallipolis Locks and Dam, which will replace Lock and Dam No. 26 below the mouth of the Kanawha River, and Locks and Dams Nos. 24 and 25 above this point, and will require all traffic to proceed through the locks. An assumption of 35% of the normal operation time has been made for lost time to cover delays at locks and lost time due to fog, wind, ice, high water, and all other causes. This is probably conservative for steady coal movements such as obtain for this general reach of the river. It will be noted that no item of profit as such has been included in the derivation. However, the fixed charges include interest at 6% on the initial amount of investment in floating plant. Depreciation is charged on a simple life basis rather than on a sinking-fund basis with compound interest. It is believed that, considering all the aforementioned refining factors in actual practice, a 2-mill rate per ton-mile for coal haul is representative and as such affords an adequate margin of profit for contract-carrier service. Representative costs of transportation by water combined with terminal differentials that are usually in favor of rail haul, afford an equitable basis of comparing the cost of transportation *via* water with the parallel service by rail, from origin to destination.

The profit and loss account for the economic analysis of the Ohio River, as made in the paper, is derived by subtracting the total annual commercial savings from the total annual charges. The following data for the Ohio River obtain for the calendar year 1934 (the latest year of the analysis):

Total freight, in tons	18 636 000
Total freight traffic, in ton-miles	1 783 924 000
Average haul, in miles	96
Savings, net, in mills per ton-mile	6.9
Savings, net, in cents per ton	66.5
Savings, net, total, in dollars	12 359 000
Charges, total, in dollars	10 058 293
Profit on improvement, total, in dollars	2 300 707
Profit on improvement, total, in mills per ton-mile	1.2

An average haul of 96 miles (based on the year 1934) on the Ohio River is equivalent to a parallel rail haul of about equal amount, at least in the upper end of the Ohio Valley. The corresponding average rail rate is about 15 mills per ton-mile for coal haul which takes a commodity rail rate. If the 15% flat increase in rail rates should finally come into effect, the increase would be 2.25 mills per ton-mile on the basis of the average haul. It is improbable that water-haul costs on the Ohio River will increase materially. If an increase of 10% is

assumed for water haul, the additional savings due to the full increase, if applicable, in the rail rates would be about 2 mills per ton-mile.

In 1905, the first calendar year of the period of analysis, the freight traffic on the Ohio River amounted to 12 773 000 tons, although at that time only two

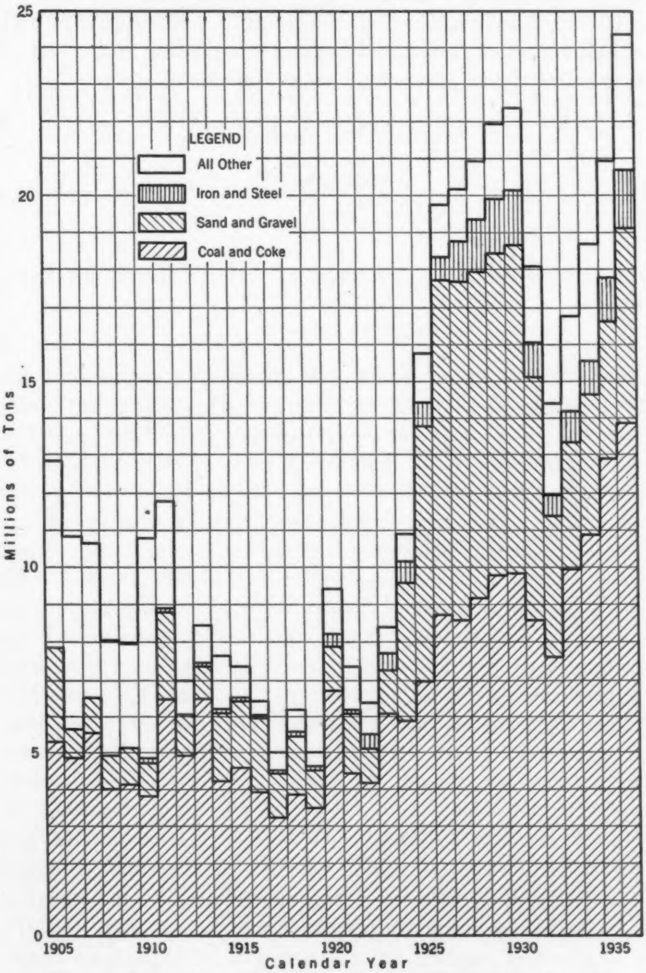

FIG. 5.—OHIO RIVER TONNAGE GRAPH, PITTSBURGH, PA., TO CAIRO, ILL. (1 TON = 2 000 POUNDS)

locks and dams—No. 1 (Davis Island) and No. 6 (Beaver)—and the Louisville and Portland Canal with its lock, were completed and in operation. Fig. 5 shows the growth of the Ohio River tonnage from 1905 to 1936, inclusive, together with the general classes of freight traffic (that is, coal and coke, sand and gravel, iron and steel, and all other freight). The impetus to the growth that occurred in the period, 1924 to 1926, inclusive, is very marked. The steady

growth from 1926 to 1930 was halted by the depression from which, however, the traffic climbed to a record of almost 24.4 million tons in 1936. It is apparent that the Ohio River improvement, commercially, not only constitutes an important part of the inland waterways of the United States, but that its field of usefulness for the transportation of basic commodities is steadily increasing. The Ohio River improvement is not the result of a passing fancy on the part of the Federal Government; nor does it belong to a passing era. The vision of the early advocates of the improvement has materialized.

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DISCUSSIONS

AERATION TANKS FOR ACTIVATED SLUDGE PLANTS

Discussion

By Messrs. Norval E. Anderson, and Weston Gavett

Norval E. Anderson,[20] M. Am. Soc. C. E. (by letter).[20a]—In his comprehensive presentation of the factors entering into the design of aeration tanks, the author notes the almost general acceptance of the 15-ft depth in the United States, but apparently is not convinced that this standardization is justified. The apparent inadequacy of data on the optimum depth of aeration tanks prompted the writer to undertake a rather extensive theoretical investigation of the question, the results of which were presented[21] in 1934. This theoretical investigation did not lead to any conclusive results. However, it is believed that a logical approach to the question was made and that some of the possibilities and limitations were evaluated approximately.

The method of approach was to set up three different theories of the possible relation between depth and air requirements, derive the necessary equations for each, assume basic operating conditions, and compute for each theory the relative air requirements for various depths.

The linear velocity theory is based on the assumption that for a given sewage, a certain minimum quantity of air (considerably less than present use) is required for oxidation, but that additional air is required to maintain a certain degree of agitation or velocity of circulation, and that this degree of agitation may be measured by the linear velocity of circulation. From this it would follow that the air required (within practical limits) is determined by the power that must be imparted to the sewage, in order to maintain a given velocity of circulation of the liquid in the tank. It can be demonstrated

Note.—The paper by S. W. Freese, M. Am. Soc. C. E., was presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 16, 1937, and published in October, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[20] Engr. of Treatment Plant Design, The San. Dist. of Chicago, Chicago, Ill.

[20a] Received by the Secretary, February 18, 1938.

[21] "The Economical Depth of Aeration Tanks of the Diffused Air Type for the Activated Sludge Process of Sewage Treatment," by Norval E. Anderson. (Thesis presented in partial fulfillment of the requirements for the Professional Degree of Civil Engineer in the Graduate School of the Univ. of Illinois, 1934.)

that on this basis the power-depth relation is a simple hyperbolic function of the form: Power per unit volume $= \frac{1}{\text{depth}}$.

The relative velocity theory, like the linear velocity theory, is based on the assumption that for a given sewage a certain minimum quantity of air is required for oxidation, but that additional air is required to maintain a certain degree of agitation or velocity of circulation. However, for this relative velocity theory, it is assumed that the necessary degree of agitation is measured by a velocity related to the tank depth. The relative velocity is assumed to be that which would give an equivalent hydraulic effect (the relation used in comparing hydraulic models), which is $V = \sqrt{\text{depth}}\ V'$. It can be demonstrated that to maintain this relative velocity, the power per unit of volume, neglecting distribution losses, remains the same for any depth.

In the contact theory the effectiveness of aeration may be measured by the area and time of contact between the air and the liquid in the tank. Furthermore, to obtain the same results from tanks of various depths, the unit of air contact area per unit of liquid volume must be kept the same. The air contact would be both that with the air bubbles and that with the atmosphere. In order to compute results in accordance with this contact theory, a number of approximations and assumptions (many of them controversial) must be made. These assumptions, all of which have some experimental basis, are as follows:

(1) Air forms bubbles of approximately the same size (about 0.01 ft) soon after emerging from the diffusers; and they remain this average size throughout the length of travel.

(2) The upward velocity of air bubbles relative to the liquid is approximately 0.8 ft per sec;

(3) The average velocity of circulation of the liquid in the path of travel in a 15-ft tank using 0.40 cu ft of air per gal of sewage, is approximately 1.7 ft per sec;

(4) The length of travel of air bubbles in the tank is approximately 1.2 times the depth of water over the diffusers; and,

(5) The atmospheric surfaces and the bubble surfaces are equally effective per unit of area, except as modified by Henry's law.

Assumption (5), although apparently contrary to the results of Mr. Freese's experiments, is supported by the experimental results of A. S. Parsons and H. Wilson[22]; J. J. Fowler and S. N. Chatterjee[23]; W. E. Adeney and H. G. Becker[24]; and H. G. Becker.[25] W. K. Lewis and W. G. Whitman[25] conclude

[22] "The Solution of Oxygen in Sewage," by A. S. Parsons and H. Wilson, *The Surveyor*, London, November 18, 1927, Vol. LXXII, No. 1896, pp. 490–494.

[23] "Note on the Aeration of Water," by Gilbert J. Fowler and S. N. Chatterjee, *The Surveyor*, London, August 12, 1927, Vol. LXXII, No. 1855, p. 139.

[24] "The Determination of the Rate of Solution of Atmospheric Nitrogen and Oxygen by Water," by W. E. Adeney and H. G. Becker, *Philosophical Magazine*, Series 6, April, 1920, Vol. 39, pp. 385–404.

[25] Absorption Symposium: "Principles of Gas Absorption," by W. K. Lewis and W. G. Whitman; "Mechanism of Absorption of Moderately Soluble Gases in Water," by H. G. Becker; and "Effect of Gas Velocity and Temperature on Rate of Absorption," by R. T. Haslam, R. L. Hershey, and R. H. Keen, *Industrial and Engineering Chemistry*, December, 1924, Vol. 16, No. 12.

that "gas bubbles * * * are particularly adapted to the absorption of the less soluble gases."

Operating data assumed as a basis for computing the comparisons are taken from approximate actual conditions at the North Side Treatment Works of The Sanitary District of Chicago, Chicago, Ill., representing rather efficient operation with a weak sewage. These assumptions are: Aeration tanks are of the circulating or spiral flow type; the water depth of the tank over the diffusers is 15 ft; the air used was 0.40 cu ft of free air per gal of sewage; the air distribution pressure loss was 1.30 lb per sq in.; the blower efficiency was 65% over-all; the average aeration period of sewage was 5 hr; and the average sludge return was 20% of raw sewage. It is further assumed that for any depth of tank the width may be approximately twice the depth.

The comparative air and power requirements for tanks of various depths, computed in accordance with the foregoing theories and assumptions, are given in Table 4.

TABLE 4.—Comparative Air and Power Requirements

Water depth over diffusers, in feet	Free Air Required, in Cubic Feet per Gallon			Brake Horse-Power per Million Gallons per Day		
	Linear velocity theory	Relative velocity theory	Contact theory	Linear velocity theory	Relative velocity theory	Contact theory
5	3.26	1.09	0.63	48.8	16.3	9.5
10	0.87	0.58	0.48	20.4	13.6	11.2
15	0.40	0.40	0.40	12.4	12.4	12.4
20	0.23	0.31	0.36	8.9	11.9	13.6
25	0.15	0.26	0.32	6.9	11.5	14.6
30	0.11	0.22	0.30	5.7	11.4	15.6

The almost total lack of direct experimental evidence makes any discussion of the relative merits of the different theories unconvincing. However, there is some indirect evidence which seems pertinent.

Consideration of the factors involved in the computations for the contact theory indicate that if they were applied to aeration tanks of the ridge and furrow type, the results would show a greater air economy than for tanks of the circulating type, since the bubbles would be retained longer in the tank, due to the absence of the circulating velocity, whereas the surface contact would be the same. Operating experience has demonstrated the opposite. This points to the predominating influence of mixing or stirring the liquid.

The importance of stirring is emphasized in the Absorption Symposium,[25] previously cited, in which Messrs. Lewis and Whitman call attention to the remarkable agreement between the results of Becker and others as to the effect of stirring on the rate of absorption, and find that this rate increases as the 0.8 power of the rate of stirring. Further evidence to the same effect is offered by the experiments of The Sanitary District of Chicago and by Karl Imhoff, M. Am. Soc. C. E., at Essen-Rellinghausen, Germany, with submerged paddle-wheels to maintain circulation, thereby reducing the quantity of diffused air required.

Of the two velocity theories, the relative velocity theory seems much the more reasonable since it gives a measure of the stirring in relation to the mass of the liquid. Furthermore, any such pronounced change in air requirements with change in depth, as indicated by the linear velocity theory, would probably have been noticed before, even with the limited opportunity for comparison.

Again referring to the Absorption Symposium,[25] it is found that the rate of absorption is dependent upon the area of interface in relation to the volume of liquid, as well as upon the rate of stirring. Since the relative importance of these factors in an aeration tank is not known, it is possible that the true power-depth relation lies between the contact theory and the relative velocity theory, although the latter probably predominates, especially for a weak sewage.

In view of the foregoing comment it appears that there probably is not much difference in power requirements per million gallons per day of sewage treated over a practical range of depths, and that more consideration may be given to relative construction costs as affected by any particular site. If the foregoing is accepted, it would modify the author's conclusions regarding depth (see text following Table 2) in two respects: (1) It is not certain that there is an increase in the cost of power with increased depth; and (2) the economical depth may be considerably more than 15 ft under less than extreme conditions as applied to costly sites or foundations.

WESTON GAVETT,[26] ASSOC. M. AM. SOC. C. E. (by letter).[26a]—The larger sewage treatment works generally have better facilities for research and investigation of plant problems than the smaller works although all experience the same difficulties. No doubt a large fund of valuable information has been accumulated at various plants that has not been published. Mr. Freese has done a useful service in presenting and interpreting data from some of the larger works.

Table 1 and Fig. 2 are interesting in indicating the optimum aeration time and percentage of return sludge for sewages of various strength as expressed by bio-chemical oxygen demand (B.O.D.). The direct relationship between optimum aeration time and percentage of sludge seemed surprising at first glance. The fact that 5-hr aeration time required 20% return sludge whereas 8-hr aeration time is definitely coupled with 50% sludge, appeared to contradict the opinion of Mr. F. W. Harris and his collaborators [27] that percentage sludge times aeration time is a constant for any given sewage. When it is realized that the values in Fig. 2 are minima for complete treatment without re-aeration of sludge, the data are not inconsistent with Harris' results. The curves in Fig. 2 may be expressed approximately in the formulas:

[26] San. and Hydr. Engr., Clyde Potts, New York, N. Y.

[26a] Received by the Secretary March 29, 1938.

[27] "The Activated Sludge Process," by A. J. Martin, Macdonald and Evans, 1927, p. 165.

$$T = 4.5 + 0.00625\ (\text{B.O.D.}) \qquad (1a)$$

$$S = 15 + 0.0625\ (\text{B.O.D.}) \qquad (1b)$$

$$S = 10\ T - 30 \qquad (1c)$$

and,

$$T = 3 + \frac{S}{10} \qquad (1d)$$

in which T = aeration time, in hours; S = percentage return sludge; and, B.O.D. = 5-day bio-chemical oxygen demand in parts per million. Mr. Harris expressed the relationship between time and percentage of sludge as:

$$S\ T = C \qquad (2)$$

in which C is a factor for a given strength of sewage. The percentage of sludge used by Mr. Harris is the percentage in the mixed liquor rather than that in return sludge as used in Fig. 2.

Assuming that C is a function of the B.O.D., the values in Fig. 2 indicate that $S\ T$ = 0.7 B.O.D. to 0.8 B.O.D., except for low values of bio-chemical oxygen demand. For a sewage with a bio-chemical oxygen demand of, say, 400 ppm, Fig. 2 indicates the minimum time of aeration as 7.0 hr and the minimum percentage of returned sludge as 40. If, however, $S\ T$ = 0.7 B.O.D., with the same 400 B.O.D. sewage, good results should be obtained with 10 hr aeration and 28% returned sludge or 6 hr aeration time and 47% sludge, although the need of re-aeration of sludge would be expected with an aeration time of less than 7 hr as shown by Fig. 2. A more general and useful expression would be obtained if the percentage of sludge were expressed (as done by Mr. Harris) as the percentage of sludge in mixed liquor after 1 hr of settling, or better as parts per million of suspended solids in the aeration tank. As the author states (see heading, "Aeration Tank Capacity"), "the percentage of return sludge, shown in Fig. 2, does not take into consideration the percentage of solids in the activated return sludge * * *." In view of the considerable variation in the concentration of solids in return sludge carried at various plants, relationships based on percentage return may be misleading unless based on sludges of similar concentration.

Experience at five plants confirms the author's statement of the advantages of diffuser tubes. The writer questions the "unevenness of air dispersion due to the fact that the air tends to escape through the top of the tube" (see "Aeration Tank Details: Diffuser Plates"). A photograph of air diffusion from a tube which appeared in a trade publication showed most of the air escaping from the top of the tube, but the tube had little water over it. When a tube is submerged 6 ft in water, the effective pressure on the top of the tube equals 6 plus tube friction and on the bottom of the tube, 6 ft 3 in. plus tube friction. The difference is slight and should not cause great difference in air diffusion. The writer believes a further advantage of tubes is the efficient distribution of air through the tank contents. Experiments at Morristown, N. J., showed that with tubes 4 ft above the bottom in a 10-ft depth of sewage, best results were obtained when the tube was located away from the adjacent wall a distance equal to one-

tenth the width of the tank. When near the wall much of the air appeared to escape along the wall without doing its share of useful work. When placed too far away from the wall the circulating velocity of the tank decreased. At the best location the air appeared to be well distributed through the sewage.

As with plates, a higher permeability was found advisable for tubes. At Mineola, N. Y., in 1928, and for Rockville Centre, N. Y., in 1929, tubes were specified with a permeability equivalent to a plate 1 ft sq with permeability of 14 to 16. Flushing by water was found useful in keeping tubes clean at Rockville Centre. Mr. Paul Molitor, Sr., cleaned tubes by a 40% nitric acid bath and heating at Chatham, N. Y.[28] Tubes at Bernardsville, N. J., were specified to pass 50 to 60 cu ft per min under 2-in. water pressure when dry, or approximately equivalent to 25 to 30 permeability for a 1-sq ft plate. At this plant only one-half the available aeration tanks are in use, so that tubes are changed once a year and allowed to rest every other year. No cleaning has been necessary since the plant was placed in operation in October, 1933. Perforated copper pipes installed at Morristown were replaced by diffuser tubes. The writer believes that the possibilities of perforated pipe have not been sufficiently investigated. When holes are perforated in the bottom of pipes which are placed above the bottom, the velocity of the rotating sewage has a sweeping effect that disperses the air bubbles effectively.

[28] *Proceedings,* New Jersey Sewage Works Assoc., 1932, p. 26.

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DISCUSSIONS

INCREASING THE TRAFFIC CAPACITY AND SAFETY OF THOROUGHFARES

A SYMPOSIUM

Discussion

BY MESSRS. HAROLD NEVIN CAREY, W. W. CROSBY, ROGER L. MORRISON, W. R. FLACK, HAWLEY S. SIMPSON, AND CHARLES M. NOBLE

HAROLD NEVIN CAREY,[24] JUN. AM. SOC. C. E. (by letter).[24a]—Some questions concerning the economic justification for expenditures for highway improvements are raised by Mr. Schmidt. He emphasizes the commonly accepted proposition that a shortening of highway length results in a savings to the individual motorist, and hence to the public in general.

Without raising the question of the proper data to use for the cost of operating an automobile, or whether any value can be claimed for savings in time, one should consider the effect on the expenses of one automobile driver of a reduction in highway length of a quarter of a mile.

The items of operating cost that may be said to be affected to any extent by the distance traveled are those for fuel, lubrication, tires and tubes, maintenance and, possibly, depreciation (see Table 13). What is the effect on these factors of shortening the distance traveled by the quarter of a mile?

Fuel.—The saving in fuel would be the sum necessary to drive a quarter of a mile, perhaps (on a basis of 15 miles per gal) 1/60 gal, or a little more than 2 oz. At 20 cts per gal this would represent 1/3 ct. It is doubtful, however, if the driver has saved that 1/3 ct when there are so many other factors affecting gasoline consumption—the adjustment and condition of the motor, the operation of the choke, and the speed of driving (a change of driving speed from 25 to 50 miles per hr may make as much difference as 4 miles per gal with a light car), whether the driving is continuous or stop and go, etc. It scarcely seems that a savings of that 1/3 ct can be claimed for the driver, inasmuch as the next month

NOTE.—The Symposium on Increasing the Traffic Capacity and Safety of Thoroughfares was presented at the meeting of the City Planning Division, Pittsburgh, Pa., October 15, 1936, and published in November, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: February, 1938, by Messrs. John A. Oakey, L. W. Mahone, Fred Lavis, Hugh A. Kelly, and Sidney J. Williams.

[24] Civ. Engr., Ayres, Lewis, Norris & May, Ann Arbor, Mich.

[24a] Received by the Secretary February 23, 1938.

or the next day a varying combination of conditions may waste more than the "saved" 2 oz of gasoline.

Lubrication.—Regardless of what data are used for miles per gallon of oil or per pound of grease, there would seem to be no monetary savings to the driver due to the quarter of a mile saved. The distance traveled has little influence on these values in comparison with such factors as whether the driver remembered to change oil at 1 500 miles or at 1 800 miles, or whether he took the car down to be greased three, four, or five times in 10 000 miles of driving.

Tires and Tubes.—In this case, again, there will be no monetary savings to the driver due to the quarter of a mile saved when his tire cost is almost entirely governed by the time when he decides to replace his tires, because tires are not used until they are absolutely worn out. A decision to buy new tires this Monday instead of next Friday has such an influence on costs as to overshadow, entirely, a saving in driving distance of a quarter of a mile.

Maintenance.—On the record, the fact that the driver did not cover the quarter of a mile could scarcely save him any money on maintenance expense.

Depreciation.—Here, again, the distance traveled has a relatively small bearing on the final depreciation cost to the driver because that is determined mainly by the time when he decides to turn in the car. Therefore, the saving in distance results in no monetary savings.

From the foregoing it would seem that it cannot be shown that, as far as the one driver is concerned, there has been any actual monetary savings due to the shortening of the distance traveled by a quarter of a mile.

The economic justification for highway expenditures based on a shortening of the distance is customarily computed by multiplying the mileage saved by the assumed affected operation costs per mile for one car and, taking this figure as the savings per car, multiplying by the annual number of cars to give the total annual savings; but if it cannot, definitely, be shown that one driver actually saves anything by the shortening of the distance, how can he be multiplied by a legion to obtain anything? Zero times *n* is still zero.

W. W. CROSBY,[25] M. AM. SOC. C. E. (by letter).[25a]—The writer's interest in the paper by Mr. McIntyre prompts the following questions:

What degree of betterment to the speed of through traffic would result from "rotary" instead of right-angled intersections?

What reduction of speed would result from "weaving" through a rotary intersection of rational diameter?

What progress has been made, and with what results on the congestion in down-town Pittsburgh, through the diversion of through traffic over the "Belt Roads"[26] around the city?

Referring to Mr. McNeil's paper the writer offers the following: His experience convinces him that many present traffic difficulties come from too narrowly weighing an improvement "from the standpoint of cost *versus* benefit to business" (see "Synopsis"). Genuine economy is a high ideal, but "benefit

[25] Cons. Engr., Coronado, Calif.

[25a] Received by the Secretary February 26, 1938.

[26] "Highway Location," by W. W. Crosby, Gillette Pub. Co., 1928, p. 127.

to business" is too mean a "yardstick." Elasticity of results, that is, the capacity for adjustment to changes in demands, convenience, and even human pleasure, is also part of the functions of highways. Mr. Schmidt refers to this under the heading, "Highway Economics."

"* * * grade separations are decidedly expensive * * *" (see heading, "Remedies for Delays Due to Cross-Interference," in Mr. McNeil's paper), and as Mr. McNeil suggests, many could be avoided (by rotary intersections, for instance) had the "elbow room,"[27] that is, wide right of way, been acquired in advance. It must be remembered constantly that the road of to-day is the street of to-morrow.

Pedestrian under-passes, unless unavoidable, will not prove efficient when they are built with steps. To encourage their use to the maximum benefits, ramps are obligatory.

Good results have been obtained in many places by requiring pedestrians, as well as vehicles, to obey traffic signals. One difficulty sometimes yet remains (referred to by Mr. McIntyre following Table 5) that of vehicular turns through "pedestrian traffic moving parallel with the traffic stream." It has been met, to some benefit, by permitting vehicular turns to the right on the red light.

Mr. McNeil states (in the paragraph preceding "Remedies for Delays Due to Medial Interference"): "It is not practicable, however, to confine any street to one-way traffic." One wonders if, for instance, the authorities of New York City would agree. He states further (see heading "Other Remedies for Delays") in effect, that an individual using his private car "spends several times the money that he would in using a mass-transportation vehicle." This situation is one to be faced squarely by highway engineers, even if deplored. He also states that unless a by-pass has certain characteristics it will fail. The writer would enlarge the list of characteristics by adding that it must have the width, or other means of protecting its flanks, sufficient to prevent an early demand for by-passing the by-pass.

Mr. Schmidt (see heading, "Highway Economics: Traffic Considerations") thinks the ownership of automobiles "surely must be approaching a 'point of saturation'." It might be well to consider the suggestions made by such an eminent authority as Frederic Law Olmstead who, in 1926, implied, that "The Saturation Point (of automobiles on roads) might depend greatly on the time available to people for using them."[28] If working hours per week become less, the writer sees no prospect for a saturation point to be reached in the near future, especially when other factors, such as the "more abundant life" and "higher standards of living," etc., are considered.

The writer wishes to commend many of the statements of the various authors in this Symposium, particularly those as to divided roadways, service ways, the elimination of parking on through-ways, the by-passing of through traffic, the use of ample diameter for circles or rotary intersections, the location of entrances to parking spaces, larger curb radii,[29] loading lanes (such as that

[27] *Roads and Streets*, October, 1934, p. 362 *et seq.*

[28] Rept. No. 39, to 5th Congress (Milan, 1926), Permanent International Assoc. of Road Congresses.

[29] "Improving Street Corners," *American City*, November, 1930, p. 140.

of Stern Brothers' Department Store on 43d St., in New York, N. Y.), the "channelization" of roadways, and the desirability of a sufficient width for the "islands" or divisional strip so that turning-traffic dangers may be minimized.

ROGER L. MORRISON,[30] M. AM. SOC. C. E. (by letter).[30a]—Of the many items discussed in the Symposium the most controversial are probably those relating to economics.

In estimating the cost of motor-car operation Mr. Schmidt uses data which were determined a number of years ago and these costs are now probably too high. A friend, who drives approximately 35 000 miles each year, kindly gave the writer detailed operating cost data on five cars of the same make which he used between January, 1927, and November, 1935. The records show a constant decrease in cost from 5.53 cts per mile on the first car to 2.80 cts per mile on the last one.

Another point, apparently not considered in Mr. Schmidt's paper, is that if the average cost per mile for gasoline, lubrication, tires, maintenance, and depreciation is, say, 3.1 cts per mile, that does not mean that a saving of 1 mile in distance gives a saving of 3.1 cts per car. The average per-mile cost of gasoline includes that used in starting and stopping, standing with motor running, and driving many short distances with a cold motor, none of which is involved, as a rule, in driving an extra mile on a trip. Furthermore, the average per-mile cost of tires includes deterioration from time, weather, injuries, and excessive use of brakes, which are not appreciable in driving an extra mile. Of course, no one knows just how much should be deducted to cover these items, but 2.5 cts per mile for passenger cars would probably be as large a value as should be used in computing savings in operating costs due to decreased mileage.

Many laymen, and some engineers, are inclined to adopt a rather contemptuous attitude toward the small per-mile savings used in the computations of highway economists. The writer has often heard it stated that such savings are too trifling to be worth considering; but in this connection it should be remembered that a gasoline tax of 1 ct per gal amounts to only about 1/15 ct per mile, and the writer has neither seen nor heard any statements that such a tax is too small to be considered. On the contrary, many a bitter fight has been waged over that particular 1/15 ct per mile.

W. R. FLACK,[31] ASSOC. M. AM. SOC. C. E. (by letter).[31a]—The most important and often the most neglected aspect of highway design is that treated in the valuable and interesting paper by Mr. Schmidt. Although the subject matter is somewhat removed from the general theme of the Symposium, the writer is most particularly concerned with the section on "Costs and Savings," which analyzes the financial investment in highway construction. One of the governing factors in such an analysis is the operating cost per mile assigned to the typical vehicle. Believing the study of this important problem might be

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[30a] Received by the Secretary March 1, 1938.
[31] Wadsworth, Ohio.
[31a] Received by the Secretary February 1, 1938.

advanced by the collection of more data, the writer desires to submit some original findings on the costs of owning and operating passenger automobiles. In Table 16, Car 1 was a 1931, two-door sedan; Car 2 was a 1931 coach; and Car 3 was a 1936 model of Car 1. Mr. Schmidt cites the studies made by Dean Agg and Professor Carter,[14] in 1928, and by Mr. Johannesson,[15] in 1931. The writer's data are based on a record of costs that began in March, 1932, and is continued to January, 1938. The values are so at variance with Mr. Schmidt's estimate that a detailed itemization and description are in order. The values shown in Table 16 are cents per mile.

TABLE 16.—COMPARATIVE COSTS OF OWNING AND OPERATING PASSENGER AUTOMOBILES, IN CENTS PER MILE

Description	Car 1, March, 1932, to March, 1934	Car 2, March, 1934, to August, 1936	Car 3, August, 1936, to January, 1938	All cars entire period	Description	Car 1, March, 1932, to March, 1934	Car 2, March, 1934, to August, 1936	Car 3, August, 1936, to January, 1938	All cars entire period
Depreciation	0.559	0.701	1.186	0.801	Greasing	0.033	0.024	0.056	0.036
Garage rent	0.365	0.364	0.187	0.309	Tires	0.090	0.103	0.004	0.068
License	0.060	0.063	0.086	0.069	Batteries	0.047	0.041	....	0.031
Insurance	0.133	0.326	0.239	0.231	Parking	0.045	0.068	0.037	0.050
Tools	0.019	0.005	....	0.009	Alcohol	0.015	0.013	0.017	0.015
Equipment	0.022	0.084	0.064	0.056	Bridge tolls	....	....	0.014	0.004
Gasoline	0.998	1.373	1.180	1.176	Fine	0.033	....	....	0.012
Repairs	0.295	0.152	0.003	0.156	....	....	....	....	....
Oil	0.116	0.058	0.081	0.086	....	....	....	....	....
Total	....	....	....	....	....	2.830	3.375	3.154	3.109
Miles	....	....	....	....	....	24 158	23 118	21 090	68 366

These cars were owned and operated in Medina County, Ohio. The average annual mileage was 11 900. About 95% of these miles were on pavement. The values for depreciation are actual for the first two cars, and those on the last car are an appraisal made from advertised prices of used cars. The garage rent is a somewhat arbitrary estimate, and is open to the question of whether it should have been included at all; nearly all houses in the vicinity had garages, and it is difficult to find a tenant for an empty garage; it is necessary for a house tenant to pay rent on a garage whether or not he has a car. The license includes tax; there are no other property taxes levied on cars in Ohio except that contained in the cost of the license; this item also includes the driver's license. The insurance covers fire, theft, property damage, personal liability, and, in 1937, collision; there was no insurance in 1932. Gasoline (various brands) was purchased retail. Oil was usually bought in 5-gal lots and added as needed; changes were made about every 4 000 miles.

No item is included for interest. These cars were regarded as necessities, and over some periods were absolutely required for the owner to reach his employment. The existence of a question as to whether or not to own a car was not recognized. A corporation may sell bonds and assume the expense of the interest on them when raising the capital for an improvement or addition

[14] *Bulletin 91*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

[15] "Highway Economics," by Sigvald Johannesson, M. Am. Soc. C. E., 1931.

to its property. It is difficult to understand how this procedure can apply to an individual in the purchase from income of what he regards as a necessity. Furthermore, as Harry J. Engel, Assoc. M. Am. Soc. C. E., has stated:[32] The theory about capital that saved money somehow automatically deserves to earn interest compounded at 6% is not well supported by the facts.

These costs are subject to adjustments to eliminate peculiarities of the conditions and to conform to more generally accepted accounting methods. The record of out-of-pocket spendings is complete and accurate.

The net operating cost per mile should include the following items from the costs given in Table 16:

Item	Cost, in cents per mile
Gasoline	1.11
Repairs	0.16
Oil	0.09
Greasing	0.04
Tires	0.07
Batteries	0.03
Total	1.50

These are the only items that will be affected directly by any reduction of distance. The gasoline cost has been reduced; the sum given in the record included considerable "stop-and-go" winter driving of the kind in which a mother takes the car out on especially cold mornings to drive the children a mile to school. This service is performed at a cost of gasoline the economist has no right to consider in examining highway relocations. Data on gas consumption, considered as a function of the distance driven per month, showed a remarkable consistency; the miles per gallon equalled about $12.00 + 2.66 \left(\frac{\text{miles driven in month}}{1\,000}\right)$. The months of low mileage per gallon and few miles per month were in the winter when starting required more than the average quantity of gas. The fact remains that the road designer is concerned with a car—or a million cars—rolling on the pavement, not as it moves, choked, in reverse gear from the garage to the street. The cost of repairs might also have been reduced, as some of the breakages had no relation to distance traveled. However, this owner was probably more fortunate than the average.

It seems to the writer that an analysis of total cost should be based upon a minimum value for the expense of vehicle operation; one should not attempt to make an economic justification of highway expenditures based on operating costs of inefficient, luxurious, or carelessly operated vehicles. As to estimating future savings, it is likely that advances in drainage, soil mechanics, design of Portland cement and bituminous concretes, and the performance of automobiles will make anticipating economies beyond a period of, say, fifteen years, imprudent. The designer might be on better ground to describe his relocation as one that would be paid for in savings to drivers over a term of fifteen years than to claim that the cost of the proposal is an equivalent 6% capitalized cost.

[32] "Social Horizons for the Engineer," by Harry J. Engel, *Civil Engineering*, November, 1937, p. 763.

The results of the two methods might be the same, but the vehicle costs can be estimated much more accurately than the interest the money might have earned.

The cost of vehicle operation may be used in estimating the expense to the public of detouring a project during construction. The writer knows of one badly timed project during which the additional operating cost of the traffic making the detour amounted to at least 30% of the cost of the construction. If this factor had been recognized some other design would certainly have been used.

HAWLEY S. SIMPSON,[33] M. AM. SOC. C. E. (by letter).[33a]—The clear analysis of the highway accident situation, presented by Mr. Vey, throws needed light on a subject of major importance. Of particular interest was his discussion of the accident experience on two-lane and four-lane highways in which he casts considerable doubt on the often reiterated theory that "congestion breeds accidents."

His data indicate that the reverse may be true when he shows that two-lane roads carrying more than "capacity" traffic have less accidents on a usage basis than when travel is lighter. Probably, however, the limited number of opportunities to make observations on the more heavily traveled two-lane roads makes data for them of somewhat less reliability than where more samples were obtained.

Furthermore, four-lane roads are shown to have a greater accident expectancy than narrower pavements. This is probably due partly to the fact that the per lane usage of the four-lane roads is less than that of the two-lane roads, so that, as Mr. Vey states, the individual motorist is allowed greater flexibility of movement, which always leads to higher accident rates. It is quite probable that if New Jersey's four-lane roads were carrying as much traffic per lane as the two-lane roads, the accident rates per vehicle-mile on the two types would not differ greatly. If this conclusion is even approximately correct, it is obvious that there is little to be gained, from the standpoint of highway safety, from the customary types of highway construction, regardless of width.

Consequently, the natural conclusion is that, unless substantial changes can be made in the aptitudes and attitudes of the driving population, which is most unlikely, safety can only be promoted by the construction of highways that will make it impossible for many types of accidents to occur.

That the expected reduction from this higher type construction will reach 75% seems to be unwarranted on the basis of available data. Although it is true that the highway which physically segregates the more important conflicting traffic flows would undoubtedly largely reduce certain types of accidents, there is as yet insufficient experience to indicate the extent to which other types of accidents may be increased in volume and perhaps even in seriousness. By removing all, or substantially all, apparent hazards, motorists may, on these new roads, be lulled into an entirely false sense of security and, therefore, attempt to obtain from their vehicles more nearly the maximum speed built into

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[33a] Received by the Secretary March 26, 1938.

them. Thus, drivers may unconsciously be led into accident-provoking situations, which they cannot, or do not, recognize until it is too late. In the higher range of speeds the ability of an individual to control an automobile decreases as the speed increases, resulting in a lessened margin between safety and accidents. If this is carried far enough, the accident toll, instead of being appreciably reduced, may be only slightly affected. On the narrow, winding, and congested roads on the other hand, the many apparent hazards induce drivers to exercise an extreme degree of caution and so to compensate, or even over-compensate, for these hazards by extra careful and prudent operation.

There are certain comparatively inexpensive measures, however, which could, if more widely used, create appreciable improvement in accident experience without such great expenditures of money as would be required to produce the "fool-proof" highway. Two of these are particularly important—highway lighting and rural sidewalks.

At speeds of the order common on rural highways, automotive headlights provide wholly inadequate pavement lighting. There are numerous limitations that seem to lie in the way of looking to the vehicle, itself, to provide sufficient light for night operation at speeds to which drivers are accustomed during the day. For a relatively small percentage of the cost of highway construction, adequate lighting can be furnished. Mr. Vey has already made studies of the effect of highway lighting on accidents and has found an economic saving in every instance. Although highway lighting is not as spectacular as the construction of high-speed, grade-separated highways, and, therefore, does not attract as much attention, economic considerations suggest that a greater proportion of highway funds be put into lighting than in the past.

Highway sidewalks, now practically non-existent, should be constructed along every mile of heavily traveled rural highway. They should be so attractive that the pedestrian will wish to use them in preference to the paved roadway. Pedestrians, as a group, have suffered greatly from the automobile; yet little attempt has been made to provide facilities that will enable them to move about with dignity and safety.

More analyses of the kind conducted by Mr. Vey, in New Jersey, should point the way to the development of a highway system with a balanced consideration of all the numerous factors involved.

CHARLES M. NOBLE,[34] M. AM. SOC. C. E. (by letter).[34a]—Highway engineers should welcome Mr. Vey's excellent paper since it presents an analysis of the value of highway improvements based on actual accident records in a State small in area but with a heavy concentrated population. The area lies between the largest city and the third largest city in the country, and forms part of a cluster of States containing less than 4% of the area and more than 25% of the population of the United States. Indeed, New Jersey is known as the "Highway of the Nation." Certainly the accident experience in such a State should include a sizable proportion of the total vehicle-miles operated in the United

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[34a] Received by the Secretary March 29, 1938.

States and should be representative of present-day congested traffic conditions as well as indicative of future trends in the less congested regions.

It is somewhat strange that highway engineers should have neglected making an appraisal of the value of each type of highway improvement from the standpoint of the accident experience of the user of the highway, particularly since it is an essential function of the engineer to deal with well substantiated facts in the design of engineering structures. It would appear axiomatic that the designer should provide a structure as ideally suited to the use that will be made of it as is practicable. The paper by Mr. Vey is a start in this direction which should prove an incentive to highway organizations to continue this form of statistical analysis in order that the use value of each type of design may be more clearly defined, and to establish its economic value. The inherent safety and "accident-proof" qualities of a design should be thoroughly established by actual tests and records before the engineer expends large sums on its construction. A design that develops a bad accident record will not reflect any credit on the designer and, in addition, it will create the impression among laymen that the engineer is either not fully alive to his responsibilities or that the problem is incapable of an engineering solution. Either conclusion would be disastrous to the highway engineer and the highway transportation industry.

The business depression years have produced a feeling among many classes of people that the highway system of the nation has been completed to as large an extent as required and that the sums collected from the motorist in the form of gasoline and oil imposts should be diverted to unemployment relief or for other governmental purposes. Indeed, large sums have already been diverted from highway funds, and some of the philosophy behind such diversion may be traced to this idea. Even among engineers the measurement of adequate highway transportation service has been computed, in the past, on the mass basis of mileage and the condition of the roadway surface rather than from the quality standpoint of the safety, comfort, and convenience of the user. Design which provides inherent safety for the motorist is distinctly a new trend. Mr. Vey's paper presents accident facts to support this trend and to substantiate the engineer's conviction that henceforth the "measuring stick" must be quality and not quantity; and, furthermore, that the country is not overbuilt but, on the contrary, that it is in urgent need of all the funds collected from motorists to modernize the existing highway system as well as to construct new trunk routes and farm-to-market roads.

It is stated that the data in the paper were derived from statements filed by the participants in accidents, with supplementary notes, in some cases, by the State Police. It is recognized, of course, that this type of data will reveal only the most rudimentary causes of accidents and will merely serve to classify the general type of accident. Mr. Vey recognizes this and rightly only attempts to utilize the latter information. The more subtle causes cannot be brought forth by participants, since it is scarcely possible for them to recognize such causes. All too often a particular set of conditions in or surrounding the highway combine to cause the motorist to commit an error in judgment which with lightning-like rapidity climaxes in an accident; and thus it is quite possible that more accidents are caused by defects in highway design than are indicated in the paper.

Nevertheless, it is interesting to analyze the information in Fig. 5 and list the percentages which certainly can be almost wholly prevented by perfected highway design; and to speculate on the approximate percentage of reduction of those types which one cannot state with absolute certainty will be wholly eliminated.

Referring to Fig. 5, it is noted that 14% of all accidents are of the right-angle type at intersections, and that 1% occur between intersections. If grade separations are constructed at all crossings with proper acceleration and deceleration lanes this first type of accident should be completely eliminated—namely, 14% of all accidents. In the case of opposite direction left-turn accidents, those occurring at intersections will also be eliminated by grade separation, namely 5 per cent. In the case of pedestrians, if their presence is removed from the motor areas and provision is made for them elsewhere, this type of accident can be almost entirely prevented; and it is safe to state that all but 2% of the total can be eliminated, namely 8 per cent. The opposite-direction accident can be eliminated by the provision of a wide space separating opposite-direction traffic, accounting for a total of 21 per cent. If all fixed objects are removed for a reasonable distance from the pavement, the fixed-object accident can be largely prevented; but in order to be conservative it may be assumed that surely one-half these accidents can be eliminated, namely 6 per cent. Many types of same-direction accidents are caused by the influence of oncoming opposite-direction traffic, and if the divided highway principle is utilized, this type of causation is eliminated. Furthermore, the use of curbs that hamper lateral freedom has an effect on this type of accident because the motorist does not have the opportunity to dodge sidewise on to the shoulder to avoid a crash in case of an emergency. Wide, smooth, stabilized shoulders, level with the pavement, permit lateral freedom and will have a favorable influence on same-direction accidents. If divided design and wide shoulders are utilized, and if it is assumed that one-half the same-direction accidents will be eliminated between intersections because of such design, then it is probable that 10% of all accidents will be prevented. The same-direction accident at an intersection is caused by the stopping of vehicles at traffic lights or slowing down or stopping to make a turn. If grade separation is utilized with acceleration and deceleration lanes these accidents will be eliminated, namely, 10% of all accidents.

To summarize, the use of known design technique will surely eliminate the following types and percentages of accidents: Right angle, 14%; opposite direction left turn, 5%; pedestrian, 8%; opposite direction, 21%; same direction at intersections, 10%; giving a total of 58 per cent. If the probable reduction of fixed object accidents (6%) and same direction between intersections (10%) is added the total becomes 74%, which is a very close check on Mr. Vey's statement of 75 per cent. Thus, it can be seen that improved engineering technique and design will have a pronounced favorable effect on accidents.

In addition to the savings resulting from the foregoing general features of design, it is very possible to reduce the accident toll still further by close attention to highway details in order that all features will conform to the requirements of the user and may be as accident proof as possible. Mr. Vey recognizes the entire problem very clearly, summing it up concisely in his paper as follows

(see heading "Synopsis"): "It is necessary that roadway facilities be designed and applied, which are inherently safe, as far as possible, in order to render the failures of human beings of lesser or little importance."

The vehicle-mile basis of comparing accident experience presents a rather accurate method of measuring comparative conditions between various general types of highways, and it is hoped that properly designed automatic recording stations will be established generally in order that such comparisons may be more widespread and that accident statistics may be compared on a more accurate basis than at present. In comparing various individual design features it may prove more illustrative, however, if some kind of rating could be established whereby each different design of the same type would be classified as being a certain percentage of a perfect, or 100%, score. In this manner, definitely bad design would very quickly score a low rating in comparison with other types of design and, being based on actual accident statistics, such a method would be of immense value in establishing correct design as well as furnishing unassailable proof to governing authorities of such good design.

The statement made in the paper that 59.6% of State highway accidents occur between intersections and 40.4% at intersections, and (see heading "Accident Location and Time of Occurrence") "that the sections of State highway having little or no cross-movements of traffic are the more hazardous" may cause some misconception of relative accident exposure. The statistics are undoubtedly correct, but it must be remembered that the actual space occupied by intersections on State highways is much less than the space traversed by a vehicle between intersections. Thus, the unit of exposure to accidents based on distance traversed, on time, or per vehicle-mile, or on any other basis of actual comparison, is much greater at intersections than on the comparatively long stretches between. As a crude illustration: If it is assumed that each intersection is 100 ft wide and that intersections are 1 mile apart, it will be thirty-six times more hazardous to traverse each foot of intersection than each foot of open highway between intersections.

It is frequently stated that each year only 15% of the drivers are the cause of 90% of all accidents, and the inference is that if these 15% were eliminated from the highways the accident problem would be solved without having to make any further improvements in the present highway system. It is true that 15% of the drivers cause a large percentage of the accidents each year, but, unfortunately, it is not the same 15% every year, except for a small percentage of accident-prone drivers who are known as "accident repeaters." If it were true that the same drivers were involved in the majority of accidents year after year they would soon destroy themselves and the accident problem would have solved itself long since, without any exertion on the part of the engineer. This suggests that the average driver is involved in the greater number of accidents—motorists who are sober and substantial citizens. In addition, the experience to date indicates that the behavior of the driver is the most difficult element in the situation to control, whereas the design of the highway can unquestionably be controlled with any degree of accuracy required. It would appear, therefore, that full use should be made of this fact, and that all the known safeguards

should be incorporated in the design to as great an extent as is economically justified in order to overcome the weaknesses and mistakes of the user.

The situation is similar to the conditions existing in industry some years ago when workmen were not protected from moving machinery and other hazards by proper safeguards, and, consequently, the industrial accident toll was quite high. At that time accidents were assumed to be an unavoidable evil and part of the risks of manufacturing, little effort being made to improve the situation. Then, as to-day in the motor field, the operator of the machine was assumed to be at fault in case of injury, the assumption being that he was inept or careless. The tremendous reduction in industrial accidents by the use of proper safeguards is an indication of the results possible when the problem is attacked with the proper mental attitude and with vision and courage. Thousands of men and women are alive and well to-day who would be dead or permanently disabled had it not been for that effort. A similar opportunity exists for humanitarian service in the motor accident field; and since, in contrast with the industrial accident problem, the highway user will pay the cost of such safeguards, he is entitled to the most perfect product which knowledge and skill can produce.

The facts and figures given by Mr. Vey form a powerful tool which highway engineers should be quick to seize in order to prove conclusively to laymen and officials that highway improvements are a vital and effective means for combating the accident problem, that the need for such improvements is imperative if the accident disease is to be checked, and, furthermore, that diversion of gasoline and oil imposts is causing the death and injury of thousands of human beings yearly. Therefore, Mr. Vey is to be congratulated on his valuable paper, and it is hoped that it will create an incentive to highway authorities to continue and extend this form of analysis in order to establish authoritatively the theory that improved highway design is a fundamental need.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ECONOMIC PIPE SIZES FOR WATER DISTRIBUTION SYSTEMS

Discussion

BY MESSRS. ELLWOOD H. ALDRICH, M. H. KLEGERMAN, AND F. KNAPP

ELLWOOD H. ALDRICH,[15] M. AM. SOC. C. E. (by letter).[15a]—In so far as it extends previous studies by Professor Camp and others into the analysis of water distribution problems, this paper should prove helpful in focusing attention on the economics of pumping and pipe installations. It sets forth a trial-and-error method to determine the economical sizes of pipe in water-works systems.

The various examples given in the paper cover, quite completely, the variable conditions that may prevail. The basic premise in all of them is that the economic installation depends upon balancing the cost of pipe line against the cost of pumping in comparative sizes of pipe. In the examples, apparently, trial-and-error methods must be utilized, first, to solve the distributional flow and then to determine the economic sizes of pipe under the assumed distribution. Quantities, depth of trench, and m, the ratio of take-off head to friction loss in the pipe (which, in turn, requires an assumption of pipe size), must be assumed and then rechecked.

Several assumptions in the paper do not appear to be in accord with current practice. Granted that pumping is required, the only variable of any practical consequence in the cost of pumping is the cost of power or fuel. Capital costs of station and equipment are unaffected by the quantity pumped or the pumping head, within the ranges commonly applicable. Likewise, operating costs, exclusive of prime power, such as supplies and operating labor, are relatively stable and are little affected by quantity or head conditions. For the purposes used herein, if it is assumed that distribution storage is adequate to equalize hourly fluctuations in flow, accuracy will not be greatly sacrificed if the cost of pumping included only the cost of fuel or power.

NOTE.—The paper by Thomas R. Camp, M. Am. Soc. C. E., was published in December, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1938, by Messrs. Clinton L. Bogert, J. M. M. Greig, Charles M. Mower, Jr., and A. C. Michael.

[15] Civil Engr., with Malcolm Pirnie, New York, N. Y.

[15a] Received by the Secretary March 9, 1938.

One of the principal purposes of storage is to permit a decrease in pipe capacity. An economic study of the distribution of water cannot neglect the cost, effective elevation, location, and amount of storage in the system.

In some localities the so-called unpredictable costs of pipe laying, such as rock excavation, wet work, removal and relaying of pavement, and avoidance of obstructions, commonly add as much as 50% to the total cost of the work. Machine excavation, when applicable, and the use of composition joints and longer pipe lengths will lessen the cost of installing pipe. These unpredictable costs are not constant but vary with the size of pipe in about the same ratio as does the cost of laying. Therefore, they will affect the economic sizes of pipe appreciably since the higher cost of pipe construction will make more economical the use of smaller pipe with increased pumping charges.

In determining the economic size of a main over a period of years, consideration must be given to the total cost over the period. For instance, in comparing total costs of two sizes of mains with increasing demand over a period of years, the smaller main will have less total cost during the early years, due to lesser fixed charges, and more during the later years, due to increased pumping costs. For a true comparison between the total costs of the two mains, the savings of one over the other in the early years is greater than would be indicated by the formula in the paper because of the effect of compound interest. Furthermore, if water sales which will result from the installation of the smaller main are sufficient to pay the total charges in the early years of its use, substantial earnings above these charges will result from increased sales during the later years. The increased charges due to higher pumping charges, therefore, will be partly or entirely offset by increased earnings. This fact will justify the use of the smaller pipe with later duplication as required. This duplication will be largely paid for by the savings due to decreased capital costs in the early years and increasing earnings throughout the period.

The paper indicates that methods for analyses of distributional flow in pipe systems and for determination of the economic sizes of pipe have hitherto not been available and that in the past designers have not had practical methods for the solution of such problems. Graphical analysis of these problems, the fundamentals of which were illustrated by the writer in 1937,[16] have been in use by him for more than ten years and have proved quite adequate for the practical treatment of these problems. The determination of the economical sizes of pipe similarly yields to simple graphical analysis.

M. H. KLEGERMAN,[17] ASSOC. M. AM. SOC. C. E. (by letter).[17a]—There is an admitted reticence on the part of designing engineers toward the use of design methods involving intricate—or apparently intricate—mathematical procedures. Accordingly, they find their attention directed, from time to time, to certain design procedures, the advantages of which are acclaimed because of their "freedom from calculus," or "because they can be solved by simple arithmetic,"

[16] "Solution of Transmission Problems of a Water System," *Proceedings*, Am. Soc. C. E., October, 1937, pp. 1511–1533.

[17] Associate, Alexander Potter, New York, N. Y.

[17a] Received by the Secretary March 19, 1938.

etc. From the foregoing viewpoint one might, at first glance, relegate Professor Camp's formulas for the design of economic pipe networks, with their derivatives, integration, and exponential equations, to the "intricate mathematical type." That conclusion, however, bears no justification, as is disclosed by a little careful examination.

Much has been written about the ease of solving flow problems, for example, by means of the Williams-Hazen formula; but certainly, the solution of this formula, prior to the introduction of special slide-rules and tables for this purpose, was by no means simpler than any of the formulas for the determination of economic pipe sizes, as presented by Professor Camp. The same might be said for most of the existing flow formulas, such as those of Kutter, Manning, etc., the use of which has been simplified by means of tables, charts, and curves, so that the computer scarcely realizes that he is using rather "complicated" formulas. Therefore, one should not evaluate a new design method solely on the basis of time involved in its solution, by comparing to the time involved for the solution of an existing method, for which, as a product of long usage, special aids have been provided.

Simplification of design methods is indeed a virtue. Such simplification, however, is sometimes brought about through approximations, and under some conditions the degree of inaccuracy in the final solution of a problem may become very much in doubt, depending on the extent of such approximations. For example, in the solution of flow problems of pipe distribution systems, it is not uncommon practice to eliminate from the computations, for the purpose of simplicity, certain of the lines that a computer may "consider" of minor mportance; or pay little, if any, attention to the effect on the flow distribution and pressure drops resulting from minor (but cumulative) take-offs. The extent of inaccuracies thus introduced is unknown. They may be insignificant under one set of conditions, but by no means so under another.

To overcome the foregoing objections, the designer is now provided with such devices as the Hardy Cross method[3] and others, the excellence and simplicity of which have already been well illustrated.[4]

Similarly, Professor Camp has presented a tool that should prove of value in the economic selection of sizes comprising pipe networks, to which past efforts have been confined to "cut and try," and "experience," due to the admitted complexity of the problem.

Selecting the economic size of a single force main is a practise that is certainly followed by every engineer. Be that main relatively long, or even quite short (of the magnitude of 500 ft) a selection of diameter is based, among other factors, upon a consideration of the annual cost of financing the construction (including pipe, jointing material, laying, pumping equipment, etc.) and of the annual energy costs, these factors being plotted against pipe diameters resulting in a U-type of curve. Certainly, if such practise is warranted in the case of force-main design, there can be no question of the value of economic

[3] "Analysis of Flow in Networks of Conduits or Conductors," by Hardy Cross, M. Am. Soc. C. E., *Bulletin 286*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[4] "Simplified Analysis of Flow in Water Distribution Systems," by J. J. Doland, M. Am. Soc. C. E., *Engineering News-Record*, October 1, 1936, p. 475.

selection in the problem of distribution system piping, which represents, in total, vastly greater length and tonnage than the average force main.

The writer believes that the method presented by Professor Camp, used in combination with an exact method of determining flow distribution, has particular merit and application in the analysis of existing distribution systems for valuation purposes. For example, in the determination of a purchase price for public utilities (and the same may be said for rate determinations) one factor at least, in which the higher Courts have been in close accord, is the principle that it is the value of the property now, for which compensation must be paid now. As a result of the widely accepted principle, that "the expenditure required to produce a property as nearly as possible like the one valued, throws an important light upon value," most water supply valuation procedure, in so far as physical plant is concerned, involves the computation of "reproduction cost." This, in turn, requires the determination of a schedule of the property and consideration of the item of depreciation.

There are many cases of the valuation of water-works properties involving principally cast-iron pipe distribution systems, wherein reproduction cost has been determined by applying unit prices which were considered to be representative at the time of the valuation, to the inventory of the property involved, and deducting therefrom an allowance for depreciation. Thus, considering cast-iron pipe alone, having determined the length or tonnage, and adopted a unit price for these items, its "present worth" (deducting depreciation) was established.

The question, however, which did not receive adequate (if any) consideration in this case was: Does the schedule or tonnage of cast-iron pipe present in the distribution system, represent the least tonnage required to serve the same streets at equivalent hydraulic capacities? If it does not, why should the purchaser be required to pay for unnecessary pipe tonnage? Particularly, if he were to reproduce the system, he would, it is properly assumed, design the most economical distribution system possible for the hydraulic conditions represented by the system being valued.

Perhaps this viewpoint is contrary to the generally accepted meaning of the term, "reproduction cost," which refers to the cost of a structure in all respects identical with the old one; nevertheless, it may safely be stated that if the enterprise being valued were completely destroyed, the chances of replacing it exactly as it was before would be remote. In other words, the worth of a new plant of equal capacity, efficiency, and durability, with proper discounts for defects in the old plant (and, of course, depreciation for use) should be the measure of value, rather than the cost of exact duplication.

The drawback to the acceptance of this viewpoint, calling, in effect, for new design, has undoubtedly been due to the fact that in the absence of analytical design procedures, engineers would differ widely in such "equivalent" designs.

It is realized that "excellence of design" receives consideration in valuation work. However, without the relatively recent methods of computing flow distribution, quickly and accurately, in various parts of the water supply distribution system, and in the absence of a method of determining the most

economical network of pipe sizes required in it, such weighting of "excellence of design" in determining value is relegated, necessarily, to the realm of opinion. To eliminate this opinion (which tends to vary with individuals) and to substitute for it an analytical method capable of identical solution by independent computers, would be a step forward in the reduction of the indeterminate factors that make valuation work controversial.

In applying the various Camp formulas to a study of an existing system (that is, for determination of least tonnage for an hydraulically equivalent system) the availability of flow records and billings, particularly in metered systems, would make possible fairly accurate estimates of take-offs. Fire flows for the determination of maximum flow conditions, as is required in some of the cases presented, could be determined from field tests. Friction coefficients could also be determined from field tests, the work being confined to representative mains in order to classify coefficients of the mains for various age groups. The extent of this determination may be carried to any degree required. However, as the author indicates, inasmuch as a reduction of the value of C, as great as from 130 to 80, will produce a variation in economic diameter of only 15%, probably typical coefficient values only need be determined by field tests. Power costs at the time of valuation can be determined definitely.

Thus, many of the factors entering into the use of the Camp formulas (as applied to analysis of existing systems) are quite capable of practical determination. Other factors, as to laying costs, etc., can probably be revised, if necessary, to apply to specific conditions.

F. KNAPP, ESQ.[18] (by letter).[18a]—There has been no good compendium on the subject of economic sizes of pipes for water distribution systems and, therefore, the paper by Professor Camp is a timely one. The author is to be commended, especially, for his attempt to classify the several cases met with in practice.

As far as the economic diameters for a gravity system are concerned, the writer understands that the method proposed by the author consists in simplifying the network so that only the larger and important mains are shown. To this series of pipes, Equation (13) or Equation (14) is applied, thus determining the best diameters. Applying this procedure to a distribution system, such as that shown in Fig. 2 (a), the mains would consist of Pipes 1, 2, 3, 4, and 5. With these diameters fixed, the sizes of the pipes of the remaining network are determined in such a manner that all, or nearly all, the available head is utilized. With the critical point at k instead of a, the procedure is repeated, but as the economic diameters for the several loading conditions will not be identical, some standard size of pipe will be selected which comes nearest to satisfying all the loading conditions.

This method assumes that the sub-branches are relatively small as compared with the main pipes. Assuming, however, that the take-offs of a network system are distributed in such a manner as to require pipes of about the same size, it is certainly not admissible to "skeletonize" the system. In this case,

[18] With the São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil.

[18a] Received by the Secretary April 15, 1938.

the flow in each pipe is known only after the pipe system has been analyzed, assuming or estimating the diameters. How would the author proceed in such a case?

The writer is unable to follow Rules (1) and (2) as formulated in the text following Fig. 2, and as the treatment of the economics of pumped supplies makes direct use of these rules, the writer hopes that the closing discussion of the author will clear up these points.

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DISCUSSIONS

EARTHQUAKE STRESSES IN AN ARCH DAM

Discussion

BY MESSRS. F. W. HANNA, A. W. FISCHER, AND RAY L. ALLIN

F. W. HANNA,[12] M. AM. SOC. C. E. (by letter).[12a]—The effects of earthquakes on dams, of both the arch and the gravity types, are important in regions where earthquakes are prevalent. The authors have presented an important element for the determination of these effects in thin circular arches of uniform thickness both with fixed and with hinged ends. The solutions presented are capable of amplification and also of extension, of which the authors are no doubt aware.

The earthquake impulse under Assumption (1) is assumed to act horizontally in a vertical plane parallel to the sides of the canyon and, therefore, parallel to a vertical plane through the crown of the arch rings of an arch barrel of the arch dam. The authors have treated only the down-stream impulse in their paper. This impulse may act either in a down-stream or in an up-stream direction.

For example let Fig. 4 represent a semi-section of a symmetrical ring of uniform thickness of a multiple-arch dam, the plane of the ring being inclined at an angle of ψ with the vertical. Let $C\,D$ be the trace, on the plane of the arch ring, of a vertical plane parallel to the vertical plane through the crown section of the arch ring. Assume the impulse of an earthquake shock to act horizontally either down stream or up stream on the arch in a direction parallel to this vertical plane. At the left of Point C in Fig. 5 is shown a down-stream impulse, dF, on an elementary volume of the arch ring, depicted in section in Fig. 4, resolved into its component forces, dR acting along CD in the arch ring and dN acting upward along the arch barrel, in the vertical plane.

Now, dR is resisted by arch action of the arch ring and dN is carried mainly by beam action of the arch ring to the adjacent piers and tends to overturn them down stream. Also, to the right of Point C in Fig. 5 are shown an up-

NOTE.—The paper by Ivan M. Nelidov and Harold F. von Bergen, Assoc. Members, Am. Soc. C. E., was published in December, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1938, by Messrs. A. Floris, and Cecil E. Pearce.

[12] Cons. Engr., Ankeny, Iowa.

[12a] Received by the Secretary February 14, 1938.

stream impulse of dF on the elementary volume and its components, dR and dN. In this case also dR is resisted by the arch ring and dN is carried mainly by beam action to the adjacent piers and tends to weaken their resistance against overturning up stream. From these considerations it is evident that

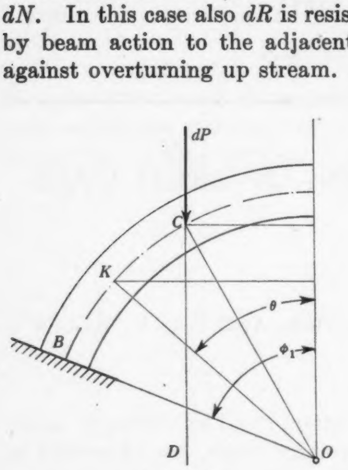


FIG. 4

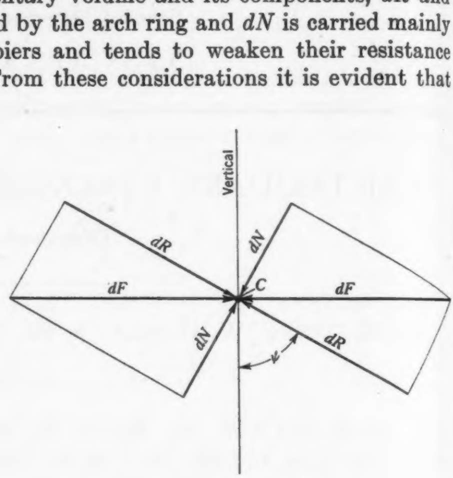


FIG. 5

the algebraic signs of dF and dR (and, therefore, of the thrust, moment, and shear in the arch ring) are reversed for an earthquake acceleration acting up stream from those of one acting down stream. It also follows, therefore, that an arch ring designed for customary water pressure down stream is well suited to resist the stresses due to an earthquake shock with a down-stream acceleration; but that an arch ring so designed may not be suitable to resist stresses due to an earthquake shock with an up-stream acceleration. Steel should be introduced to take the tensional stresses produced by this latter kind of shock.

In the case of an earthquake acceleration parallel to the canyon walls, the force, F, in Fig. 5 will be different for each arch ring of an arch barrel depending on its position relative to the bottom of the arch. Neglecting the effects of the piers or canyon walls, the value of F for each arch ring may be computed approximately by the formula used for buildings[13] or for vertical slices of gravity dams.[14] For multiple or single arches relatively high for their lengths, the effects of the piers or canyon wall abutments are of considerable consequence. Impulses conveyed to a high single arch in a narrow canyon may even exceed that conveyed to it by the canyon floor and may be so great as to prevent the proper use of the building formula for finding the impulse force, F, for arch rings of different elevations. The stresses in such an arch would be more nearly approximated by considering it to be a plate supported on three edges.

In the case of an empty reservoir, the effects of earthquake shocks must be applied to the masonry of the structure only; but for a full or partly full reservoir the influence of the movement of the reservoir water on the masonry stresses must be taken into consideration. This subject has been treated in

[13] "Reinforced Concrete and Masonry Structures," by Hool and Kinne, p. 600.
[14] "The Design of Dams," by Hanna and Kennedy, p. 109.

previous publications[15] and is referred to here only for the purpose of making the category of forces caused by earthquake shocks complete.

The authors have limited their considerations to thin circular arches, but the principle used in their analyses may readily be applied to thick circular arches and also to arches of variable thickness, both of the symmetrical and unsymmetrical types, merely by substituting the earthquake load for the usual water pressure and then following the analysis otherwise applicable. In this, as in all cases, it is best to avoid complications by assuming that r' is equal to r. The distance from the center of gravity of the arch ring is,

$$r' = \frac{\int_{r-\frac{t}{2}}^{r+\frac{t}{2}} r\, dr\, r\, d\phi}{\int_{r-\frac{t}{2}}^{r+\frac{t}{2}} dr\, r\, d\phi} = r\left(1 + \frac{t^2}{12\, r^2}\right) \dots\dots\dots\dots\dots (31)$$

and, therefore,

$$\int \frac{r'}{r} = 1 + \frac{t^2}{12\, r^2} \dots\dots\dots\dots\dots\dots\dots\dots\dots (32)$$

For $r = 2\, t$, $\frac{r'}{r} = 1 + \frac{1}{48} = 1.002$, which is negligible in the face of the general uncertainties of the entire problem.

Solution of Equation (31).—Integrating,

$$r' = \frac{\int_{r-\frac{t}{2}}^{r+\frac{t}{2}} r\, dr\, r\, d\phi}{\int_{r-\frac{t}{2}}^{r+\frac{t}{2}} dr\, r\, d\phi} = \frac{\frac{d\phi}{3}\Big[\, r^3 \,\Big]_{r-\frac{t}{2}}^{r+\frac{t}{2}}}{\frac{d\phi}{2}\Big[\, r^2 \,\Big]_{r-\frac{t}{2}}^{r+\frac{t}{2}}} \dots\dots\dots\dots\dots (33)$$

Canceling $d\phi$ and substituting limits,

$$r' = \frac{2}{3}\,\frac{\left(r^3 + \frac{3}{2} r^2\, t + \frac{3}{4} r\, t^2 + \frac{t^3}{8} - r^3 + \frac{3}{2} r^2\, t - \frac{3}{4} r\, t^2 + \frac{t^3}{8}\right)}{\left(r^2 + r\, t + \frac{t^2}{4} - r^2 + r\, t - \frac{t^2}{4}\right)}$$

$$= \frac{2}{3}\,\frac{\left(3\, r^2\, t + \frac{t^3}{4}\right)}{2\, r\, t} = \frac{2 \times 3\, r^2\, t\left(1 + \frac{t^2}{12\, r^2}\right)}{3 \times x\, r\, t} = r\left(1 + \frac{t^2}{12\, r^2}\right)$$

and, therefore:

$$\frac{r'}{r} = \left(1 + \frac{t^2}{12\, r^2}\right) \dots\dots\dots\dots\dots\dots\dots\dots\dots (34)$$

Let r in the right member of Equation (34) equal $2\, t$; then $\frac{r'}{r} = 1 + \frac{1}{48} = 1.002$.

[15] "The Design of Dams," by Hanna and Kennedy, pp. 109–113; and "Water Pressures on Dams During Earthquakes," by H. M. Westergaard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 98 (1933), pp. 418–472.

A. W. FISCHER,[16] ESQ. (by letter).[16a]—The subject of earthquake stresses is one of much importance to every engineer who designs structures in localities where earthquakes may occur, and the authors have contributed a valuable paper for calculating certain stresses in an arch dam.

The subject of earthquake stresses in an arch dam either hinged or fixed is treated at quite some length on the assumptions made by the authors and on their basis of calculations. The maximum stresses (as given in Table 1) are equal to $\pm$ 104 lb per sq in. for the arch with fixed ends. For the arch with hinged ends the maximum stress, as given in Table 1, is 27 lb per sq in., compression. The writer doubts if the stress given for the hinged-end condition is the maximum that could occur from an earthquake. Neglecting rib-shortening and assuming that due to an earthquake the abutment is displaced only 0.08 ft outward, and then substituting the values given in the numerical example (with E = 3 000 000 lb per sq in.), the fundamental equation for the horizontal thrust, on a slice 1 ft high, in a segmental two-hinged arch with a constant, I, is,

$$H = \frac{3\,000\,000\ (144)\ (1/12)\ (0.08)\ (19)^3}{(171)^3\ [(51.5/57.3)\ (2 + \cos 103°) - 1.5 \sin 103°]} = 29\,500 \text{ lb}$$

For the particular arch the moment at the crown equals 29 500 (171) vers 51° 30′ = 1 904 000 ft-lb, and from this moment the stresses at the crown equal $\pm$ 220 lb per sq in. The stress from direct load equals 11 lb per sq in., tension. The stresses are based on the assumption that the arch is a monolith and has no vertical expansion joints.

Authorities might question the possibility of the abutment moving 0.08 ft, but the writer believes that in some localities this could happen and in extreme cases it could be greater than 0.08 ft, and stresses due to slight movements of the abutments should not be overlooked. The stresses in an arch dam with fixed ends will not be as large as in a two-hinged arch for the same displacement of the abutments; but in all arch dams, either two-hinged or with fixed ends, the stresses due to slight movement of the abutment should be considered.

RAY L. ALLIN,[17] ASSOC. M. AM. SOC. C. E. (by letter).[17a]—The earthquake effects on the stresses in an arch dam are clearly illustrated in this interesting paper.

Since, ordinarily, no tension can exist between a rock abutment and a concrete arch unless special anchorage has been provided, the cases noted by the authors must be for an arch, loaded prior to the earthquake shock with an intensity of compression stress greater than the intensity of tensile stresses indicated in the author's Table 1; or they must have assumed an equivalent compression distribution that will give the same moment and thrust at the abutment as those determined by the authors.

Where no arch thrust exists prior to earth shock, such as for an empty reservoir with horizontal arches, the case where the acceleration is acting

[16] Care, Construction Service, Veterans Administration, Washington, D. C.

[16a] Received by the Secretary April 7, 1938.

[17] Hydr. Engr., Bureau of Eng., Public Utilities Comm., San Francisco, Calif.

[17a] Received by the Secretary April 8, 1938.

parallel to a line through the abutments is similar to that where the arch is stood on one end with a body force equal to that due to earthquake acceleration. This case would place all the reaction at the abutment where the earth shock is transmitted to the dam, and would cause stresses of twice those values given by the authors at the corresponding abutment; and, at the crown, it would cause a direct thrust equal to one-half the abutment thrust, and a bending moment equal to the product of this crown thrust and its corresponding lever arm normal to the line through the center of gravity of each half of the arch. This case is not as vital to safety as that of a dam with full reservoir; yet it may be necessary to stiffen a slender arch to satisfy this condition.

Those engineers interested in the design of arch dams are indebted to the authors for their contribution on this subject of earthquake stresses in arch dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GRAPHICAL REPRESENTATION OF THE MECHANICAL ANALYSES OF SOILS

Discussion

BY MESSRS. JOEL D. JUSTIN, L. B. OLMSTEAD, T. A. MIDDLEBROOKS, AND FRANK E. FAHLQUIST AND WALDO I. KENERSON

JOEL D. JUSTIN,[21] M. AM. SOC. C. E. (by letter).[21a]—The author presents an interesting method of classifying soils. The semi-log scale, shown in Fig. 1, is in quite general use for plotting the results of mechanical analyses. Some technicians, however, plot with the small sizes at the left and the larger sizes at the right as the author has done in this case, whereas others plot with the large sizes at the left and the small sizes at the right. This seems like a small matter, but when one becomes familiar with the meaning of curves plotted in a certain manner and another technician appears, who insists on doing it differently, it takes some time before the curves mean very much.

There would seem to be no sound reason why the semi-log scale and its ratio for plotting results of mechanical analyses, as well as the direction of plotting, should not promptly be standardized. The writer does not care which way it is done as long as every one does it the same way.

The author proposes a soil classification based on two factors: (1) The mean grain size, the size than which 50% of the material by weight is smaller; and (2) a gradation factor based on the slope of the curve as more specifically defined by the author.

Such a method of classification would have its advantages, because a soil for which the mechanical analysis curve had a certain slope and a certain "mean grain size" would have fairly uniform physical properties common to other soils falling into the author's same classification.

The writer doubts, however, that the classification proposed by the author would have any practical advantage over the Kendorco classification and

NOTE.—The paper by Frank B. Campbell, Assoc. M. Am. Soc. C. E., was published in December, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1938, by Messrs. Donald M. Burmister and A. J. Weinig, Jr.; and March, 1938, by Messrs. Carl H. Kadie, Jr., Carlton S. Proctor, T. T. Knappen, Jacob Feld, and Howard F. Peckworth.

[21] Cons. Engr., Philadelphia, Pa.

[21a] Received by the Secretary February 23, 1938.

particularly over the modified Kendorco[22] classification as utilized by the U. S. Army Engineers of the Providence, R. I., District, and elsewhere.

In addition to any system of classification, such as the Kendorco, or the one advocated by the author, it is desirable to use a very general classification so that names, such as clay, silt, sand, and gravel, may be applied to soils with particle sizes that fall between certain limits. The so-called M. I. T. classification is satisfactory for this purpose.

The Kendorco classification has been used by the engineers of the Metropolitan District Water Supply Commission of Massachusetts and also in modified form by the U. S. Army Engineers of the Providence, R. I., District. Under this plan soils are classified by numbers. Even numbers indicate soils of relatively uniform grading, and odd numbers represent soils of relatively variable grading. Coarse soils are represented by the lower numbers and fine soils by the higher numbers. Thus, 2 in the modified Kendorco classification represents a uniform coarse, to medium, sand and 12 a uniform clay.

Synthetic samples representing the different classes (from 1 to 13) of soils may be made up. With a little experience one can, quite successfully, determine by visual inspection into what classification a given soil will probably fall. As soils falling into any given classification have quite uniform physical characteristics, and permeabilities are known within reasonable ranges in advance of tests on the actual samples, the idea has definite practical advantages.

The writer considers the modified Kendorco classification to be one of even greater practical usefulness to the engineer who must deal with soil as an engineering material, but who is not himself a soils technician. He hopes that some one of the engineers who have developed this method of classification to its present degree of usefulness will embrace the opportunity afforded by this discussion to describe it more fully.

L. B. Olmstead,[23] Esq. (by letter).[23a]—In the mechanical analysis of soil materials texture is expressed as percentages of several size classes. Numerous attempts have been made to increase the usefulness of these data by varying the size limits so that some simple mathematical relationship may exist between the diameters, areas, or volumes of the particles in the different classes. Others have attempted to increase the usefulness of existing data by various forms of curve plotting. Mr. Campbell has attempted both.

In addition to semi-logarithmic plotting of size limits against summation percentages, Mr. Campbell has picked out three points on the graph, presumably at such positions as he believes will usually furnish the best criteria of the shape and position of the curve. These points are called diameter-grade designation and grade-line deviation. Whether this contribution by the author succeeds in increasing the application of the mechanical analysis data will depend upon the use that will be made of it. Thus far, little use has been found in the laboratory of the U. S. Bureau of Chemistry and Soils, for derived data, or

[22] "Permeability Determinations, Quabbin Dams," by Stanley M. Dore, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 682–711.

[23] Physicist, Div. of Soil Chemistry and Physics Research, Bureau of Chemistry and Soils, U. S. Dept. of Agriculture, Washington, D. C.

[23a] Received by the Secretary February 26, 1938.

for graphical representations of mechanical analysis data. This is due in part, no doubt, to the fact that the chemical nature of the finer soil material, as well as the size distribution, affects the physical properties.

In attempting to draw conclusions from mechanical analysis data, it is necessary to take account of the validity of the data. Determinations made by the hydrometer and the pipette methods may not check, and sieves used in different laboratories for the same size limit may differ by more than 50% in area of mesh opening.

In his plan for size classification, Mr. Campbell proposes to adopt the limits, but not always the same grade names, used by the International Society of Soil Science for materials between 2 microns and 2 mm. He extends the range at each end of the scale, and subdivides each class once. Since, in this classification, the integers, 2 and 6, recur alternately, the ratios between class limits are 3 and $3\frac{1}{3}$, alternately applied. There are two other rhythms holding fast to the International size limits that might be considered if their practicality is ignored. One involves the ratios of 4 and 2.5, alternately applied. The class limits would be 0.2, 0.8, 2, 8, 20, 80, * * * microns. For most soils the separation at 8 microns is quite likely to give a more even division of the silt class than when the separation is made at 6 microns. Since the logarithms of 2 and 8 are 0.301 and 0.903, respectively, the mantissa differences are almost exactly 0.60 and 0.40, a circumstance that facilitates semi-logarithmic plotting. This plan is open to the same objection as that proposed by Mr. Campbell. It provides too many classes of the small grains and apparently too few of the large ones.

The other and more flexible system would retain the decimal classes of 2, 20, 200 and 2 000 microns, of the International System, and extend it at both ends of the scale, as Mr. Campbell proposes. Then, in regions where the number of size classes are inadequate for the needs of a particular laboratory, additional optional classes might be inserted, based upon the ratio of $\sqrt{10}$ between limits. If further separations should be required, the ratio $\sqrt[3]{10}$ or $\sqrt[4]{10}$ could be used, depending upon the number of sub-divisions needed.

This classification possesses a rhythm, which the mathematical mind appears so emphatically to demand, and makes a flexible provision for a sufficient number of size classes; but is it workable? There are no means of separating clastic materials strictly according to size. About all that can be done is arbitrarily to define equivalent diameters in terms of settling velocities and sieve specifications. If screening values are to yield equivalent sizes close enough to satisfy this classification, special screens, no doubt, will have to be provided. Even if the necessary special sieves might possibly be obtained, the selection of a classification which requires them is open to serious objection.

The writer suggests modifying this plan, in so far as it is necessary, in order to use standard or other accurate and easily obtainable screen materials. Sieves having screening values nearest the nominal sizes of the extended International Classification would be adopted and always used in the region where they apply. Highway engineers probably would omit the smaller sized groups and agronomists the larger ones. In addition, supplementary intermediate classes, utilizing standard screens, should be set up for use when more size classes are needed.

If conventional size designations are desired for the sieved material, they must be defined arbitrarily in terms of mesh openings, because the screening values of clastic materials varies with the shape factor. The present lack of accepted sieve specifications partly nullifies the benefits to be derived from the International Classification. In the smaller sized classes the relation of particle size to settling velocity permits sub-divisions of the material at almost any point. However, there is no fixed relation between settling velocity or particle size and the readings at stated time intervals of the soil hydrometers in common use. These hydrometers were originally calibrated for one particle size distribution, and their design is such that they cannot detect some unusual distributions no matter how many class limits are provided.

The fact that many institutions would not adopt a standard size classification of soil materials at this time is no reason why such a standard should not be set up and made available for all who are free to use it. Conditions may arise that permit laboratories to make gradual changes from the classification in use to a standard system. Much work and many years were required for the adoption of a classification by the International Society of Soil Science, and that classification should be used if possible as the basis of an amplified system.

The size limit of 2 microns appears to have been well chosen by the International Society. Accurate determinations cannot easily be made upon all kinds of soil materials below this limit, with the methods of dispersion and analysis now available. It is difficult to make satisfactory determinations even at this point upon some lateritic soils. In the U. S. Bureau of Chemistry and Soils the upper size limit of extracted colloids is not far from 0.2 micron, but the separation is seldom carried to completion. The upper limit for clay is now set at 2 microns, the material between 2 and 50 microns being classed as silt. An additional pipette sample is now taken for a 20-micron class limit, and a sieve separation is made at 0.2 mm, so that the results of analysis can be expressed in either the Bureau, or the International, Classification.

T. A. Middlebrooks,[24] Assoc. M. Am. Soc. C. E. (by letter).[24a]—With all due respect to the author for his conscientious attempt to obtain uniformity in soil classification, it is believed that more could be done by withdrawing a number of classifications now in use, rather than recommending new ones to confuse the field further. Little can be gained by additional soil classificationsthat vary only slightly from standard accepted classifications. The classification offered by the author does not vary in any important features from the most widely used classification of the U. S. Bureau of Chemistry and Soils. Therefore, why change?

Grain-size dimensions in the silt and sand ranges are arbitrarily set and have no particular significance as to the soil characteristic. Whether the division between sand and silt is set at 0.05 mm or 0.06 mm is of no consequence. Engineers merely need time to become familiar with the classification agreed upon. On the other hand, the division between silt and clay is of great importance and it cannot be set satisfactorily on any particular grain size, but

[24] Senior Engr., Board of Engrs. for Rivers and Harbors, Washington, D. C.

[24a] Received by the Secretary March 18, 1938.

must be determined by mineralogical examination for the different types of clays encountered. Rock flour, which is finely ground rock mineral, may have a large percentage of grains within the clay range of grain sizes, but it will not have the characteristics of a clay mineral. The writer has found montmorillonite clays in which the division between silt and clay is approximately 0.005 mm and bedilite clay in which this division is approximately 0.002 mm; therefore, it is apparent that no definite limit can be set for clay.

Experience of the writer in various sections of the United States has shown that either the Massachusetts Institute of Technology, or the U. S. Bureau of Chemistry and Soils, classification is applicable for all types of soil provided the division between silt and clay is determined by mineralogical examination.

FRANK E. FAHLQUIST,[25] ASSOC. M. AM. SOC. C. E., AND WALDO I. KENERSON,[26] ESQ. (by letter).[26a]—A new method for graphically presenting the grain-size distribution of a soil is proposed by the author. He also submits a soil classification based on the mean grain size and gradation of soil particles. Acceptable standards for describing the grain-size distribution of soils are needed, and the author should be commended for presenting his ideas to the profession. The use of such expressions as uniform coarse sand or uniform fine sand conveys definite impressions of particular types of uniformly textured soil. However, the use of such expressions as sandy silt, silty sand, clayey silt, silty gravel, clayey loam, etc., conveys an inadequate conception of non-uniformly graded materials. Such terms are often interchanged and incorrectly used by engineers. This results in vagueness and considerable confusion.

The Bureau of Chemistry and Soils, U. S. Department of Agriculture, has devised a classification that has been in general use for many years. More recently, the so-called Massachusetts Institute of Technology (MIT) soil classification has been used increasingly among engineers and soil technicians. Both these classifications are included in Fig. 1 of the paper. The writers prefer the MIT classification because it is easier to remember and is more logically constructed in that successive decrease in arbitrary grain-size limits follows a definite pattern. However, neither the MIT nor the Bureau of Soils classifications are adequate for describing non-uniformly graded materials correctly. Furthermore, in analyzing fine-textured sediments, such as silts or clays, there is difficulty in recognizing the limiting sizes of each type of material. Soil scientists do not agree where the line should be drawn in separating colloids from the coarser fractions. Notwithstanding this fact, there appears to be no reason why arbitrary limits, such as are shown in the MIT classification, should not be used. The writers believe that use of the terms, "clay" or "fine silt," for instance, should be based not only on a grain-size analysis, but what is much more important from an engineering viewpoint, on the behavior characteristics of the material.

The author proposes to classify soils according to a mean diameter size (grain-size diameter at 50% ordinate) and five arbitrary grade lines as shown

[25] Geologist, War Dept. at Large, U. S. Engr. Office, Providence, R. I.

[26] Chf., Soils Testing Laboratory, U. S. Engr. Office, Providence, R. I.

[26a] Received by the Secretary April 4, 1938.

in Fig. 1. This is essentially an improved adaptation of the soil constants, "effective size" and "uniformity coefficient," introduced by the late Allen Hazen, M. Am. Soc. C. E. These new constants partly fulfill a long desired improvement growing out of appreciation by engineers of the inadequacy of Hazen's coefficients to classify soils properly.

A soil classification based on grain size only, however, is not sufficient. Engineers who have investigated the finer grained sediments have long recognized this point. Charles Terzaghi, M. Am. Soc. C. E., has stated:[27]

"The properties that determine the behavior of the soil in the foundation pit directly are not the uniformity, nor the mineralogical composition, nor the water content. They are:

"(1) The volume change produced by an increase of the pressure acting on the soil; * * *.
"(2) The permeability of the soil; * * *.
"(3) The cohesion or the shearing resistance of the soil under zero load; * * *."

The extent to which these properties are affected by the mineral and water content and natural structure must be determined in the soils laboratory before final classification is made.[28] In addition, a soil classification to be of practical use in the field must be adaptable to reasonably accurate classification of natural sedimentary deposits.

The Providence (R. I.), District, U. S. Engineers, has developed a soil classification which meets the requirements imposed by conditions in the field and by the behavior characteristics of the sediments. During the two years, 1936-1937, this idea has been fully tested and found reliable. More than 5 000 mechanical analyses performed in the Soils Laboratory have proved its adequacy both as to gradation and size limitation of the individual classes. It is based, but with important modifications, upon the Kendorco classification described by Stanley M. Dore, M. Am. Soc. C. E.[29] Rigidly standardized descriptive terms are used in the Providence classification as follows:

Class	Description of Material
1	Clean Gravel: Contains little coarse to medium sand.
2	Uniform Coarse to Medium Sand: Contains little gravel and fine sand.
3	Variable—Graded from Gravel to Medium Sand: Contains little fine sand.
4	Uniform Medium to Fine Sand: Contains little coarse sand and coarse silt.
5	Variable—Graded from Gravel to Fine Sand: Contains little coarse silt.
6	Uniform Fine Sand to Coarse Silt: Contains little medium sand and medium silt.

[27] "The Science of Foundations—Its Present and Future," by Charles Terzaghi, *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 296.

[28] *Loc. cit.*, p. 297.

[29] "Permeability Determinations, Quabbin Dam," by Stanley M. Dore, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 682–711.

Class	Description of Material
7	Variable—Graded from Gravel to Coarse Silt: Contains little medium silt.
8	Uniform Coarse to Medium Silt: Contains little fine sand and fine silt.
9	Variable—Graded from Gravel to Medium Silt: Contains little fine silt.
10	Uniform Medium to Fine Silt: Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10C	Uniform Medium Silt to Coarse Clay: Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	Variable—Graded from Gravel or Coarse Sand to Fine Silt: Contains little coarse clay.
12	Uniform Fine Silt to Medium Clay: Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12C	Uniform Clay: Contains little silt. Possesses behavior characteristics of clay.
13	Variable—Graded from Coarse Sand to Clay: Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13C	Variable Clay: Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

Soils are divided into thirteen general classes. Distinctions between soil classes are based upon grain-size limits as shown graphically in Fig. 4. An important feature to be noted in this diagram is the flexibility allowed for the limiting diameters of fine silt and coarse clay. Either of these materials is allowed to range between 0.006 mm and 0.0006 mm. This flexibility is introduced by the writers in recognition of the fact that natural depositions of fine-textured sediments, such as glacial silts (rock flour), behave differently from clays despite similarity of grain-size distribution. It is interesting to note that this same flexibility is recognized by the Society's Committees on Road Materials, as shown by the author in Fig. 1.

Grain-size limits of the even numbered classes have been chosen arbitrarily so that the particle diameters at the 90% and 10% ordinates conform to the limiting sizes as recognized in the MIT classification. Investigations in the laboratory have disclosed that grain-size curves of variable depositions vary over a wider range in their upper parts as compared to their lower parts when attempts are made to conform to this classification. Therefore, the idealized and limiting curves for Classes 3, 5, 7, 9, 11, and 13 are discontinued at the 50% ordinate. Heavy full lines indicate idealized depositions, whereas broken lines indicate limiting boundaries within which classified depositions may vary. Sediments which do not conform to either of the two principal idealized slopes can be easily classified by this system. For instance, a sediment can be classi-

fied as Class 7–5. This designation means that the upper part of the curve lies within the Class 7 range and the lower part within the Class 5 range. Likewise, a Class 5–7 material indicates an extremely variable soil in which the upper part of the curve lies within the Class 5 range and the lower part within the Class 7 range. This same procedure is applied to variations in gradations of uniformly textured soils. Thus, a Class 6–4 or Class 4–6 is recognized.

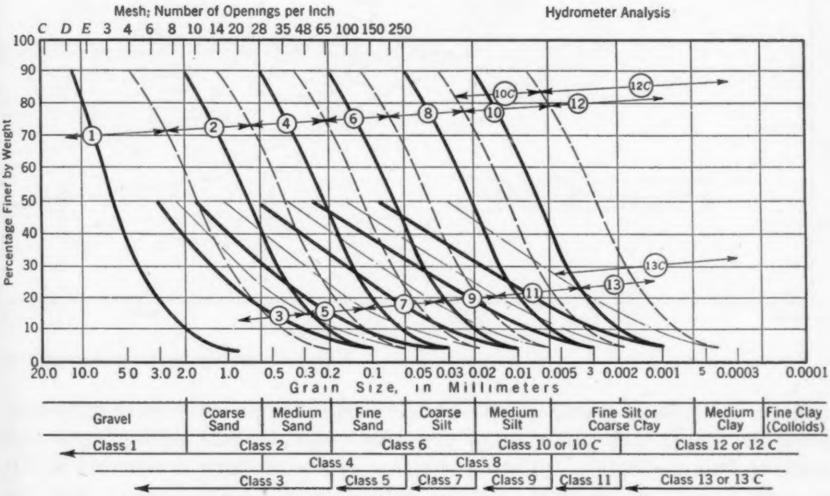


FIG. 4.—DIAGRAM SHOWING LIMIT OF SOIL CLASSES

There is surprising conformity between the author's grade lines as shown in Fig. 1 and the several possible gradations recognized in the Providence classification: The author's Grade 1 conforms very closely to the slope of the idealized curves for Classes 1, 2, 4, 6, 8, and 10; Grade 2 conforms to the slope of variations in the foregoing sediments recognized as Classes 2–4, 4–6, 6–8, 8–10, 10–12; Grade 3 conforms to the Providence Classes 5–3, 7–5, 9–7, 11–9, 13–11; Grade 4 conforms to Classes 3, 5, 7, 9, and 11; and Grade 5 conforms to Classes 3–5, 5–7, 7–9, 9–11, and 11–13.

It is to be noted that the Providence classification does not rely upon certain soil constants, such as the 50% mean diameter in Mr. Campbell's classification or the grade lines, as obtained from the 20% and 80% sizes. Instead it utilizes the entire mechanical analysis curve. This is accomplished with the aid of a transparent sheet of celluloid on which is inked a full set of classification curves similar to those in Fig. 4. The master plat is placed and oriented over an unclassified grain-size curve. The class range into which this curve falls is immediately apparent and is recorded directly on the curve sheet. This method makes for rapidity and accuracy.

The behavior of fine-grained sediments particularly general Classes 10, 12, and 13 is markedly different when considered from a regional standpoint. Thus, in the glaciated New England area, fine-grained sediments derived by glacial abrasion and occurring in glacial lake deposits are found, by tests, to be

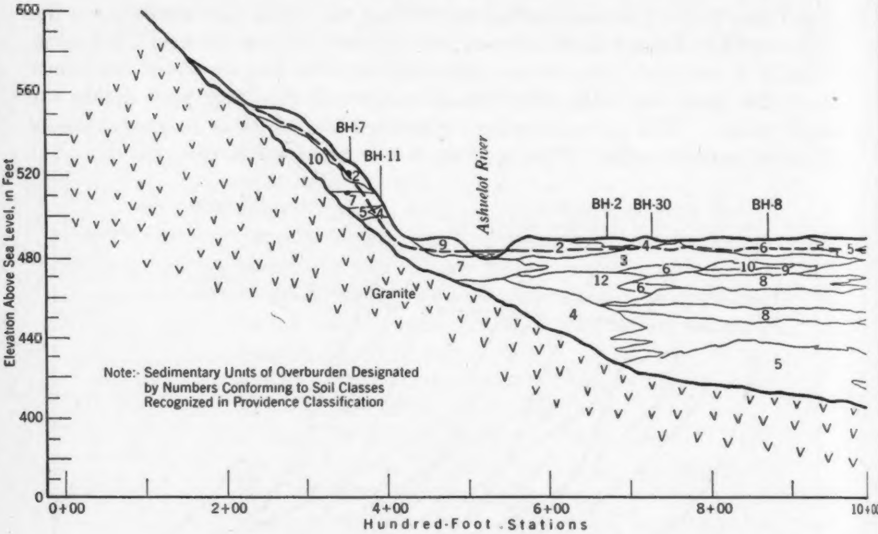
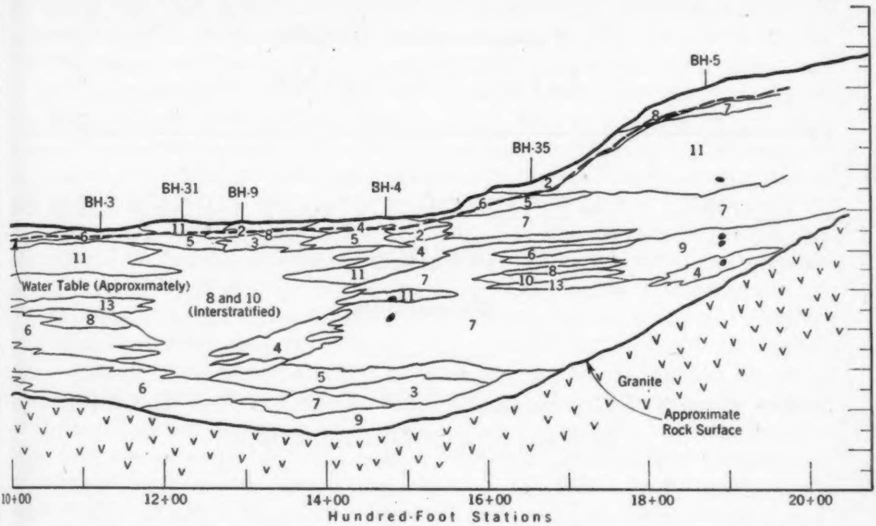


FIG. 5.—PROVIDENCE CLASSIFICATION

angular in shape of grain, capable of being consolidated rapidly, slightly compressible (depending partly upon the mica content), and possessing a high angle of internal friction. These sediments can be properly designated as silts or rock flours. In contrast, alluvial sediments of similar fineness from the Mississippi Valley have decidedly different properties. These sediments are often composed of flat and scale-like clay particles, together with coarser fractions derived from rock decay. They consolidate slowly, are highly compressible, and possess a low angle of internal friction. It is readily seen that to classify these materials, as Class 10, for example, without further qualification would lead to misunderstanding on many important and critical properties. In such cases the Providence classification provides for referring to the former as Class 10, and to the latter as Class 10C, the letter C indicating behavior characteristics of clay.

The author's statements as to the geologic significance of the mechanical analysis curve are valuable. In the past, the geologic aspect of soils has been too little appreciated by engineers and soil technicians. The ease with which soils classified under the Providence System can be indicated in geologic sections is shown in Fig. 5. The character of the material in a formation and the mode of occurrence are generally reliable criteria as to the geological history of that formation. Certain sedimentary types are more prominently developed in one section of the country than in another. For example, glacial deposits are prominently developed in the northern part of the United States and Canada. Loess deposits, probably of wind-blown origin, occur throughout a large part of the Mississippi Valley, particularly in Nebraska, Wisconsin, and Arkansas. Soils in the South and Southwest are largely of residual origin. In river valleys throughout the United States there are large accumulations of alluvial deposits



APPLIED TO GEOLOGIC SECTION

of both fine and coarse texture. A few examples of the accumulations in certain recognized geologic formations are:

Geology	Classes
Alluvial deposits (coarse-textured)	1, 3, 5, 7, 9, 2, 4
Alluvial deposits (fine-textured)	4, 6, 8, 10, 10C, 12, 12C, 11, 13, 13C
Glacial deposits—till	9, 11, 13, sometimes Class 7
Glacial deposits—outwash	1, 3, 5, 7, 2, 4, 6
Glacial lake deposits	6, 8, 10, 10C, 12, 12C
Aeolian (wind-blown) deposits	4, 6, 8
Loess deposits	Chiefly Class 8
Residual soils	3, 5, 7, 9, 11, 13, 13C

The author's classification has considerable merit. His paper comes at an opportune time, and it is felt that it will result in considerable clarification as to acceptable definitions of soil separates.

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DISCUSSIONS

ECONOMIC ASPECTS OF ENERGY GENERATION A SYMPOSIUM

Discussion

BY MESSRS. PAUL E. GISIGER, F. A. DALE, RAY S. QUICK, F. KNAPP, AND D. S. JACOBUS

PAUL E. GISIGER,[45] M. AM. SOC. C. E. (by letter).[45a]—The paper by Mr. Sporn offers a welcome clarification of the essential points pertaining to the cost of electrical energy. Although the facts and figures undoubtedly give an excellent and clear picture of the distinct elements of which the cost of power is made up, it may be questioned whether some of the conclusions drawn are not a shade too pessimistic as far as the future of hydro-electric power generation is concerned.

In the technical literature of the last fifteen or twenty years predictions of the decline of the proportion of hydro-electric power in the total power supply of the United States can often be found, but such predictions have not been confirmed by later statistics. In 1920, hydro-electric plants produced 37.1% of the total energy generated by public utility central stations. In 1924, this percentage was 33.8, and, in 1933, it was 40.7; in the 12-month period ended October 31, 1937, it was 35. These figures show variations governed largely by the annual variations in river flow, but as a general trend, it appears that water power has kept its own during a period when the efficiency of steam-generating apparatus has greatly increased and the annual output of central stations nearly doubled. One factor in this is probably the spreading recognition of the fact that the combination of hydro and steam power is very often more advantageous than the use of either one alone. In this regard, it may be noted that, in 1920, among the large cities on the Atlantic seaboard, Baltimore,

NOTE.—The Symposium on Economic Aspects of Energy Generation was presented at the meeting of the Society and at the Joint Meeting of the Power and Engineering-Economics and Finance Divisions, Pittsburgh, Pa., October 14, 1936, and published in the December, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: February, 1938, by Messrs. W. S. Finlay, Jr., R. M. Riegel, Ralph Bennett, J. M. Mousson, Ben C. Sprague, V. M. Marquis, R. L. Sackett, D. J. McCormack, and I. E. Moultrop; and March, 1938, by Messrs. M. M. Samuels, and E. W. Kramer.

[45] Structural Engr., Pennsylvania Water & Power Co., and Safe Harbor Water Power Corp., Baltimore, Md.

[45a] Received by the Secretary February 18, 1938.

Md., was the only one which received large blocks of hydro-electric power; at present (1938), Washington, D. C., Philadelphia, Pa., Providence, R. I., and Boston, Mass., are also supplied to a considerable extent with energy generated in hydro-electric plants. To cite also a foreign example of the same trend, the Metropolitan Area of Paris, France, which lies much nearer to coal fields than to water-power sites, was, until 1927, supplied to less than 1% with hydro-electric power while in 1936 the proportion of hydro-electric energy was 36 per cent.

The writer cannot agree fully with Mr. Sporn's statements that hydro-electric plants, in general, as a source of primary power, are out of the economic range, and that as a general rule no hydro-electric plant is self-sufficient. These opinions hold only within geographical and other limitations. They are undoubtedly correct for large sections of the United States, but their application to other parts of the world—for instance, to Canada—would be out of place. If a hydro-electric plant can be built at some figure near the average of the costs quoted in Mr. Sporn's paper, within a reasonable distance from the places where its output is to be used, its economic justification depends ultimately on the greater or lesser variations of stream flow. In the northeastern section of the United States these variations are, in most cases, of such magnitude that only by the most careful analysis, taking full account of all limitations and possibilities which depend upon stream flow and its improvement by storage as well as of the manner in which the energy can be fitted into the daily and annual load curve of a distribution system (or to what extent it can replace steam energy), is it possible to determine whether development of a given water-power site is economically justified, that is, preferable to steam power?

On the other hand, if a hydro-electric plant can be built at, say, $140 per kw, and the stream flow allows it to be operated at a daily and annual load factor of not less than, say, 50%, it is not difficult to show that steam power at its present, or even a higher, efficiency will cost more to generate, and a substantial margin remains to cover the cost of transmission. It is true that in the Northeastern United States there are few water-power sites that satisfy such requirements, but next door, in Canada, they are the rule rather than the exception. Consequently, steam plants produce less than 5% of all the power generated in Canadian public utility plants.

With the foregoing, it is intended to show that, for the present at least, the relative economy of steam as against hydro-electric energy depends to such an extent on place and circumstance that no general statement can be made declaring one to be cheaper, unless the specific conditions are given to which it applies.

One point which appears to be overlooked sometimes when costs of steam and hydro-electric power are discussed, is the influence of the prevailing rates of interest on the relation of these costs. It is evident that, since capital cost is normally more than 90% of the cost of hydro-electric power, whereas the cost of steam power is made up to only about 50% of capital charges, a change in interest rates will affect the cost of hydro-electric power more than the cost of steam power (see Fig. 37, in this connection). The figures on which Fig. 37

is based correspond to cost figures used in Mr. Sporn's paper. It is assumed that the kilowatt of installed hydro-electric capacity costs $160 and the kilowatt of installed steam capacity, $100. The annual cost of operation of the hydro-electric plant is taken at 75 ct per kw of capacity and all other charges exclusive of interest at 5% of cost of installation. In the case of the steam plant, the cost of operation is taken from Fig. 30 of Mr. Sporn's paper for an annual load factor of either 45.6% or 57.0%, corresponding to either 4 000 or 5 000 hr of operation per yr. All other charges exclusive of interest are assumed to be 7% of the cost of installation, making the capital charges for the steam plant 2% higher than for the hydro-electric plant, as assumed by Mr. Sporn. No transmission charges were added in case of steam energy, while in the case of hydro-electric energy, such charges have been added for a transmission distance of 50 miles. Transmission costs for this distance and for the load factors used were taken from Fig. 32 of Mr. Sporn's paper. They were added without change to the kilowatt-hour cost resulting from a 5% interest rate, while for other interest rates they were increased or decreased in proportion to the total cost exclusive of transmission.

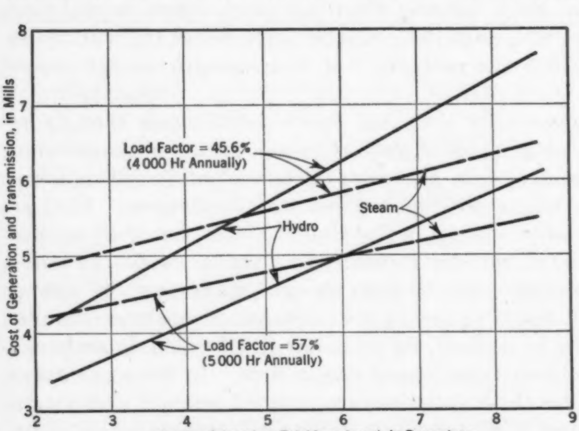

FIG. 37.—INFLUENCE OF INTEREST RATE ON COST OF ENERGY

The resulting curves of total cost per kilowatt-hour show plainly that a reduction in interest rates is relatively more favorable to hydro-electric power than to steam power and *vice versa.*

F. A. DALE,[46] M. AM. SOC. C. E. (by letter).[46a]—The cost of generating electrical energy by means of falling water involves four major factors: (1) The cost of the development; (2) the quantity of water available; (3) the efficiency at which the plant is capable of converting the water into energy; and (4) the efficiency at which the plant *does* convert the water into energy. Mr. Sporn has emphasized the fact that the preponderant portion of total hydro-electric energy costs is the fixed charges on investment. Mr. Rogers has presented a masterly summation of what the designers have accomplished in furnishing equipment which is capable of converting a very high percentage of the water energy into electrical energy.

Additional emphasis should be placed on Factor (4) in the cost of generation, namely: That the final cost per kilowatt-hour depends to no small extent on the

[46] Hydro-Elec. Engr., Nepsco Services, Inc., Augusta, Me.

[46a] Received by the Secretary March 2, 1938.

proportion of possible kilowatt-hours actually generated. From the point of view of cost of development, or fixed charges on investment, the investment cost for delivering water to the power plant may be 75% of the total cost of the project. The water, therefore, is certainly not free. Unlike steam plants, for which fuel can be purchased as it is used, the administration of a hydro-electric plant must purchase, in advance, the entire fuel supply for the life of the project. The water should be used as carefully and effectively as if it were being weighed out and paid for by the ton. Dividing fixed charges by kilowatt-hour output gives fixed costs per kilowatt-hour. The cost per kilowatt-hour can be decreased just as effectively by increasing the denominator of this simple fraction as by decreasing the numerator.

The designers and manufacturers have certainly done their part in furnishing plants that can convert the maximum of available energy. Unfortunately, they cannot follow through to any great extent and insure that their products will be used most effectively. Mr. Rogers mentions automatic load control devices that have been developed to insure efficient operation and which are extremely effective where applicable. The largest part of energy generation, however, is still dependent on the knowledge and ability of the operator. Much can be done to improve the results obtained by operators, especially in the smaller plants, by equipping the plants with additional devices to aid the operators. The major gains, however, must come through the initiative and demands of operating management.

Possibly, as much as a billion kilowatt-hours of energy each year slip, unused, through existing plants in the United States. The corresponding annual increase in returns would be about $5 000 000. Saving this energy is a problem which engineers will solve if given the opportunity.

Ray S. Quick,[47] Esq. (by letter).[47a]—The interesting paper, presented by Mr. Rogers, covers the development of hydro-electric energy, including illustrations of actual machines which have made it possible to convert hydraulic power efficiently into mechanical energy for driving electrical generators.

One of the most interesting problems developed in the generation of hydro-electric energy on the Pacific slope has been an effective means of controlling water-hammer in comparatively long and high-pressure penstocks. Surge tanks are used where possible, but they do not provide an effective control near the power-house where pressures are high. The penstocks are of heavy construction and constitute a major part of the first cost of the development; therefore, their economical design is a problem of first importance. Where water economy is not required, the simplest means of controlling pressure rise is the synchronous by-pass. With this arrangement, the flow through the unit is constant and independent of load. The hand-operated needle of the impulse turbine with governor-operated deflector is a characteristic example. Most plants, however, depend upon seasonal storage, thus requiring a high degree of water economy, and it is here that the pressure regulator, or relief valve, finds its most valuable application.

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[47] Chf. Engr., The Pelton Water Wheel Co., San Francisco, Calif.

[47a] Received by the Secretary March 14, 1938.

In Fig. 4, Mr. Rogers has illustrated two typical forms of automatically controlled nozzles for impulse turbines, arranged for full water economy. The left-hand view shows the governor-operated power needle with auxiliary relief by-pass, whereas the right-hand view shows the governor-operated deflector with a slow-moving, power-needle follow up. With the first mentioned arrangement, the oil-pressure governor controls the opening of the power needle to suit the load. An oil-filled dash-pot is interposed in the linkage connection between the power and the relief needles in such a manner that a rapid closing movement of the power needle causes a corresponding or synchronous opening of the relief needle. The heavy closing springs on the dash-pot urge the relief needle to return to the closed position at a rate determined by the dash-pot adjustments and resulting in very little pressure rise. An opening movement of the power needle resulting from an increase in load first synchronously closes the relief needle if it is still open, otherwise simply by-passing the oil in the dash-pot and leaving the relief needle shut. As it is necessary to accommodate oncoming load with the relief needle shut, it is customary to adjust the rate of opening of the power needle to such a value that there is little pressure drop in the penstock and, consequently, no objectionable subsequent pressure rise. A sudden opening of an orifice on a high-head penstock otherwise would produce, first, a sudden pressure drop, followed by an excess pressure. These sudden surges are objectionable and must be prevented.

In the case of the needle-deflector combination, the deflector moves quickly on load rejection and bends the jet away from the water-wheel buckets. The power needle follows up at a rate permitted by the retardation of the water column in the penstock. Its rate of closure, therefore, is of the same order as that of reclosure of the relief nozzle of the aforementioned alternate system. Likewise, the rate of opening of the power needle with the deflector combination is at the same rate as the corresponding opening of the power needle of the relief nozzle system. It is thus evident that either method of controlling an impulse turbine results in complete water economy with gradual load changes and with wastage of water only for a short period, such as 1 min to 2 min following a sudden loss in load.

When they are wide open, the relief nozzles, illustrated in the foreground of Fig. 2 of Mr. Rogers' paper, discharge a jet about 8.5 in. in diameter and have an output of approximately 35 000 hp under a 2 200-ft head. The potential destructive force of this jet is enormous. The safe conversion of this energy into turbulent flow in the tail-race has constituted a major problem of impulse turbine design. Special forms of baffles have been used in certain applications with success. The most effective form, however, is the through type of energy absorber, consisting of a circular open-bottomed bowl having a cone supported within it by radial ribs. The jet of water is concentric with the point of the cone, which acts to spread the water radially against the inside of the bowl, which, again, turns it upon itself and allows the discharge to mingle with the tail-race water without objectionable external disturbance. The energy absorber at Big Creek No. 2-A is a comparatively small structure.

In the case of the Francis turbine an outstanding installation of a pressure regulator and through type energy absorber will be found in Units N5 and N6 at Boulder Dam. Fig. 5 of Mr. Rogers' paper shows the shop assembly of these two turbines, the outlet for one pressure regulator connection being shown in the foreground. In this case, the pressure regulator is of the needle type using a pilot system inter-connected with the gate-shifting ring through a dash-pot control, and having a hydraulically operated main needle or plunger with mechanical interlock. The mechanism is designed so that should the

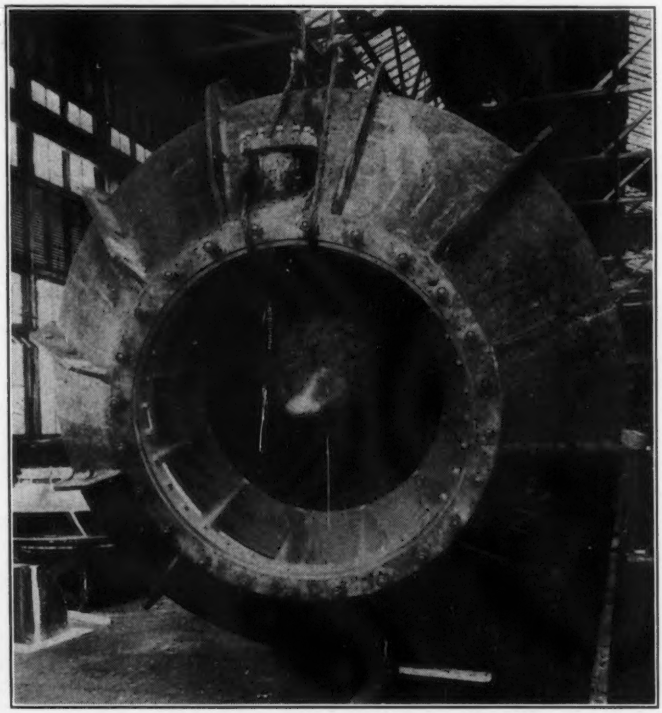

FIG. 38.—SHOP ASSEMBLY OF ENERGY ABSORBER

movement of the main plunger be obstructed the governor will be able to close the turbine gates only at the rate permitted by the dash-pot timing, thus preventing objectionable pressure rise in the penstock even under such an abnormal set of conditions.

Under average conditions, each pressure regulator will have a capacity of 1 860 cu ft per sec under a 480-ft head which corresponds to more than 81 000 hp in the jet when computed at 80% efficiency. The energy of this water must be absorbed in order to prevent an objectionable disturbance in the tail-race. Fig. 38 shows the shop assembly of the energy absorber for the Boulder pressure regulator. The jet of water (which is approximately 52 in. in diameter) strikes directly upon the cone supported in the bowl, turning the water

radially outward. The water is then turned back upon itself beneath the bowl from which it discharges downward as turbulent flow through a discharge elbow and concrete-lined horizontal outlet into the tail-race. The entire absorber is of heavy cast-steel construction, with ample provision for the admission of air into those regions near the jet where air may be required.

The pressure regulator and energy absorber system constitute a valuable contribution to the economical generation of hydro-electric energy, making possible a reliable means of controlling the speed and pressure rise with a penstock of minimum first cost. The pressure regulator also assists, in a most valuable way, in damping out periodic swinging surges that may be established under certain conditions of governing with a closed penstock system.

F. KNAPP,[48] ESQ. (by letter).[48a]—The paper by Mr. Rogers is decidedly not cosmopolitan. Only hydro-electric installations in the United States are discussed, and the only reference made to plants abroad is to the Shannon development in Ireland. The progress made in Europe (especially in the past few years) is of such magnitude as to be of special interest to American hydraulic engineers, but is entirely set aside by the author.

As far as impulse turbines are concerned, the largest unit—with 78 600 hp for a static head of 2 360 ft—is running in a Brazilian plant.[49] The 49 500-hp Pelton turbines for the Dixence plant in Switzerland, set the record of head—5 770 ft.

There is a marked tendency to build large impulse turbines with vertical shafts and two nozzles. Typical examples are the Etzel and the Handeck plants, both in Switzerland. The straight-flow nozzle design, already incorporated in the turbines of the Dixence plant, is an old Swiss idea, introduced just prior to the World War by V. Gelpke.

Considerable difficulty was experienced for a time with cavitation of impulse-turbine buckets, and it developed that a design successful for a head of, say, 1 300 ft, was a failure for a head of 2 300 ft. Stroboscopic observation of the buckets in motion, and refined design methods, were the means of overcoming these troubles. Efficiencies of impulse turbines are now 90% or more.

The 32 600-hp Francis turbines of the Piottino plant, in Switzerland, with a speed of 750 rpm, work under a net head of 1 060 ft, and the "back-pressure" turbine of the Zapello power station in Italy operates under the unusually high head of 1 280 ft, with 33 ft of back pressure. Each of the two sets installed develops 8 800 hp at a speed of 1 500 rpm.

Designs have been prepared for a two-stage turbine, with a maximum capacity of 37 000 hp and a total head of 1 280 ft, running at 1 000 rpm. The first stage consists of a Francis runner, designed for a head of 1 100 ft; and the second stage is a propeller runner, for a head of 165 ft, with a back pressure of 15 ft. The last part of the head would be used in a single and separate turbine of the propeller type. It is clear that such an arrangement diminishes considerably the size of the units.

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[48] With São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil.

[48a] Received by the Secretary February 19, 1938.

[49] "Water-Power in Brazil, with Special Reference to the São Paulo Development," by A. W. K. Billings, M. Am. Soc. C. E., *Minutes of Proceedings*, Inst. C. E., London, 1936.

It is only right to mention that the development of the high-speed turbine of the propeller or Kaplan type would not have been possible without the simultaneous advance in the design of draft-tubes, inasmuch as in these turbines the water still has about 30% of its total energy, when it leaves the runner, and this energy must be regained in the draft-tube. The elbow draft-tube of the Safe Harbor turbines (Fig. 17), is typical of this development; the other types of energy regainers, such as the hydraucone and the spreading draft-tube, are now almost obsolete—mostly for economical reasons. It is interesting to note that the elbow draft-tube, as used to-day, was first developed in the United States.

As far as laboratory testing of complete hydraulic turbines is concerned, it is noteworthy that one Swiss manufacturer has installed an aerodynamic research laboratory, using air instead of water. Besides the easier testing and the smaller space requirements this research station offers the great advantage that it is now possible to measure directly the pressures on the rotating runner blades.

The writer was surprised at Mr. Roger's statement that it is not yet possible to determine the actual benefit obtained by machining the blades as regards either performance or avoidance of pitting. It seems that with the great relative velocities in high specific speed turbines there should be a certain gain in efficiency with correctly machined blades. At least, one practical case known to the writer indicates that a correct shape of the blades, such as that obtained by machining, is of decided advantage in avoiding local cavitation.

The description of the unit installed in the Marmet plant, as given by Mr. Rogers, is most interesting. A few European installations have turbines with runners of the Kaplan type, but with fixed guide-vanes of the diagonal type. The efficiencies obtained were between those of a Kaplan and a propeller turbine.

The efficiencies of the model pump-turbine unit shown in Fig. 21 are excellent and rather promising. A unit of this type, with a Kaplan runner and diagonal guide-vane, has been installed in the Baldeney plant, in Germany.

D. S. Jacobus,[50] Esq. (by letter),[50a]—Such a true picture of the developments that have occurred in the generation of energy are presented by Mr. Orrok that the writer will simply add some comment concerning changes in steam boiler practice. His oldest recollection in power generation is of the Corliss engine installed in the Machinery Hall of the Centennial International Exhibition held in Philadelphia, Pa., in 1876. There was an iron platform surrounding the engine at a somewhat higher level than the floor of the building and the writer can remember standing on this platform at the time of the Exhibition. What awed him most was the tremor of the platform, and the noise of the gears and, above all, the dignified motion of the engine and its great size.

The Centennial engine consisted of two beam engines of 700 normal hp each, although they could be worked up to a total of 2 500 hp if required. The two engines were connected to a common fly-wheel, which was 30 ft in diameter, with gear teeth on its outer face. The fly-wheel made 36 rpm. Steam was

[50] Advisory Engr., The Babcock & Wilcox Co., New York, N. Y.

[50a] Received by the Secretary March 25, 1938.

furnished by twenty Corliss upright boilers of 70 hp each, the steam pressure ranging from 25 lb to 80 lb per sq in., according to the amount of power required.

A modern 50 000-kw steam turbine would generate about fifty times as much power as the normal amount developed by the Centennial engine; it could be run with steam from one or two large steam boilers in place of twenty boilers; and the entire unit would occupy less space.

The late Professor Thurston, who is the first authority mentioned by Mr. Orrok, was an indefatigable worker and was noted for the exceptional accuracy of his statistics. He was an able teacher and, in addition to foreseeing the tremendous increase in power development, he impressed upon his students the necessity of dealing with the human side of engineering. In addition to specializing in steam engines he did much in contributing to the safety and improvement of steam boilers. He often referred to the tests on steam boiler explosions by Francis B. Stevens who was instrumental in having the railroad companies subscribe toward the expense of the tests. Professor Thurston served as a member of the Federal Commission that witnessed Mr. Stevens' tests. The boilers were tested in 1875 on the United States Army Reservation at Sandy Hook, N. J. A bomb-proof retreat was built for observers. Four old steamboat boilers and five new parts of boilers built for the tests were fired until the the pressure either reached a point at which they failed by leakage or they exploded. Explosions had often been attributed to some mysterious action; these tests, however, showed that they were due to simple and preventable causes.

In addition to contributing to safety in boiler construction, Professor Thurston did much in revealing the fundamental features bearing on efficiency, in papers and books that he wrote on the subject.

Advances in the art of boiler construction have involved improvements in the furnaces and fuel-burning equipment as well as in the boiler proper. Water-cooled furnace walls have greatly reduced the time required for shutting down boilers for inspecting, cleaning, and repairs. The use of pulverized fuel has grown to a point where it is utilized in most new equipment for large coal-fired boilers in the United States. Steam pressures and temperatures of superheated steam have increased to a point at which boilers for central station and industrial plants with natural circulation are being built for 2 000-lb working pressure and more; and, forced-circulation boilers for experimental purposes are in use, that can be run at 4 000-lb pressure, which is greater than that corresponding to the critical temperature. The steam temperatures have increased correspondingly. A comparison of central power-plant boilers in 1905 and to-day is given in Table 7.

Improvements in instruments and apparatus contributed much to the attainment of present-day economies. Engineers of the "old school" will remember the uncertain results secured by the so-called barrel calorimeter which was recommended for use in the first report of the Committee on Steam Boiler Trials of the American Society of Mechanical Engineers. Professor Alfred M. Mayer, who for many years was the head of the Department of Physics at Stevens Institute of Technology, at Hoboken, N. J., and who wrote a number of

important papers on his tests of inaudible sounds, etc., observed the writer operating such a calorimeter. After standing for a time without saying anything, he turned away with the remark, "Why not use a wooden horse bucket and a tin back thermometer?" After the barrel calorimeter, the so-called coil calorimeter was used. It was not until the introduction of the throttling calorimeter that results were secured of sufficient accuracy to determine the effect of different collecting nozzles in the steam main.

TABLE 7.—COMPARISON OF CENTRAL POWER PLANT BOILERS IN 1905 AND 1938

Descriptive	1905	1938
Heating surface of boiler proper, largest unit, in square feet	6 040	53 926
Heating surface of boiler, superheater, economizer, and air heater, in square feet	7 000	144 402
Working pressure, in pounds per square inch	225	650 to 2 500
Temperature of superheated steam, in degrees Fahrenheit, maximum	550	700 to 950
Maximum evaporation, in pounds per hour per boiler unit	30 000	1 000 000
Volume of boiler setting for largest boilers, in cubic feet	7 650	42 255
Height from bottom of walls of setting to center of steam and water drum for largest boilers, in feet	19	80

In the early Eighties tests of an alternating-current and a direct-current electric generator were made at the Stevens Institute for Mr. Edward Weston. There was a conspicuous lack of accurate instruments. A home-made "tangent galvanometer" was used for an ammeter for measuring the direct current, calibrating on each test by depositing copper. The "tangent galvanometer" consisted of a single circle of copper wire placed in a vertical plane, at the center of which was an ordinary magnetic compass. A voltmeter was checked by measuring the resistance of the circuit; and, a standard silver cell was built for checking voltage measurements. Both the "tangent galvanometer" and the voltmeter were affected by local magnetic influences. Finally the "tangent galvanometer" was removed to a small shed built on the campus well away from the main buildings, after which its readings were more consistent.

The troubles encountered in measuring the alternating current were still greater than in the case of the direct current and a number of methods of checking were devised. It was the extreme difficulty of obtaining reliable measurements in these tests that led Mr. Weston to develop the accurate electrical measuring instruments for which he became famous.

Much study has been made of boiler-fed water conditions. A modern steel-tube economizer requires the elimination of oxygen and carbon dioxide from the water, and necessitates the use of apparatus unknown in the older practice. The feed-water in most large central stations is as pure as the distilled water secured from a drug-store. With the higher degree of purity, troubles have developed through corrosion and corrosion fatigue of the steel, which necessitate adding chemicals to the water. Certain elements in the feed-water may lead to a characteristic cracking of the steel plates, known as caustic embrittlement, and it is necessary to provide a certain balance between the elements in the water to avoid this action. The treating of feed-water has become an art in itself, and, in order to operate an up-to-date power plant to the best advantage, the chemist must co-operate with the engineer to a greater extent than in the past.

Improvements in boiler-room auxiliary apparatus have contributed much in co-operation with boiler improvement to attain the present-day economics. The efficiency based on an accurate determination of the heat of combustion of the fuel has replaced the evaporation per pound of fuel or per pound of combustible, in reporting tests of boilers. The old-time boiler-room, where nearly everything was manually operated, has now yielded to what is more nearly an automatically, well-conducted laboratory, with a few men watching the controls of the factors bearing on combustion, steam generation rate, pressure, and temperature. Fuel feed, air supply, and fuel air ratio, boiler-feed, water level, and all pressures are controlled automatically with connections to an operating board. Its gages and meters showing the steam flow, percentage of carbon dioxide, and steam flow-air flow are symbolic of the scientific management independent of manual action, which has eliminated wastes that are bound to occur when operators must act on mere indications to guide them in making adjustments. These same elements also insure prompt response to changing load conditions and add to the safety of operation.

A marked advance has been made in the ability to predict the efficiency obtainable with the great variety of arrangements and sizes of furnaces for different fuels, boilers, superheaters, economizers, and air heaters, and the approximate rules in general use a comparatively few years ago have now (1938) been replaced by new formulations of a much more exact nature.

The Boiler Code Committee of the American Society of Mechanical Engineers has been an important factor in advancing the art and the safety of boiler construction. There are now 23 active members of the Boiler Code Committee and 40 members of the Conference Committee which is made up of representatives of those in charge of enforcing the Code in the States and municipalities that have adopted it. More than 160 men in all are continually interested and take a regular part in the work of the Boiler Code Committee, and many others having special information render invaluable help. Since it issued its first Code in 1914, the Code Committee has been successful in securing unanimous decisions in all its final actions.

Practically all steam and water drums for high-pressure boilers are now fusion welded, the old practice of riveting being correspondingly replaced. For a number of years the Boiler Code Committee was asked to formulate rules for boilers and other pressure vessels for what was at first known as autogenous welding. Certain very restricted rules were agreed on for unfired pressure vessels and for seal welding of certain joints in steam boilers. After many hearings, rules were issued in 1931 for fusion welding drums or shells of power boilers.

Radiographic examination by the X-ray or gamma ray in making nondestructive tests of the welds was the turning point in the development of fusion welding because, through its use, the safety of the welds is less dependent on the human element. The rules for fusion welding in the Boiler Code are still far from complete and are being gradually extended. This involves much painstaking work, since the use of steels and alloy steels of higher tensile strength has introduced problems requiring solution on which the best metallurgists are not yet in entire agreement.

In dealing with economy in the generation of electrical energy, one should not overlook the important saving accomplished by electrically tying in one generating plant with another and in this way insuring a continuous service with a reduced capital investment. This was one of the factors that enabled electricity to be supplied to the consumer without increased cost during the period of high prices in spite of increase in the cost of labor and fuel; and it is an invaluable aid. However, it is evident that if carried too far there may be a major shut-down due to an accident in one or more of the generating plants or on a transmission line; so there is a point beyond which it will be dangerous to go in effecting a saving by not installing spare units in a given generating station. How nearly this point has been reached, through the natural effort to avoid further investments in the present period of uncertainty as to the near future, is something of vital public interest.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STREAM POLLUTION IN THE OHIO RIVER BASIN A SYMPOSIUM

Discussion

BY MESSRS. KARL IMHOFF, HENRY D. JOHNSON, JR., CHARLES M. REPPERT, EDWIN K. MORSE, JAMES H. LE VAN, AND JOHN C. H. LEE

KARL IMHOFF,[30] M. AM. SOC. C. E. (by letter).[30a]—An impounding reservoir or a sewage lagoon is a natural purifying arrangement for sewage or polluted streams.[31] In Germany, many plants of this type, both large and small, are operating satisfactorily. Purification takes place according to the same rules as in the self-purification of streams. It is possible to calculate the oxygen demand so that the amount of oxygen used can be equalized with re-aeration. In making such a calculation, various factors must be taken into account:

(1) The consumption and the absorption of oxygen depend on the detention period and on the temperature. Absorption is governed, in addition, by the rate of oxygen demand.[32] Complete biological purification may be assumed to take place in 20 days detention at 20° C.

(2) Oxygen absorption is also proportional to the surface area of the pond. The greater the surface and the less the depth of the pond, the greater will be the oxygen absorption for any given volume and detention period.

(3) For all depths of more than approximately 2 ft, the depletion of oxygen and the speed of decomposition are constant. For depths of less than 2 ft, the speed of decomposition increases rapidly with decreasing depth, probably because the plant and animal life on a lighted bottom has a stronger effect. In very shallow ponds the biologic purification is completed in a few days.

NOTE.—The Symposium on Stream Pollution in the Ohio River Basin was presented at the meeting of the Sanitary Engineering Division at Pittsburgh, Pa., on October 14, 1936, and published in January, 1938, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: February, 1938, by Messrs. D. E. Davis, L. S. Morgan, C. A. Holmquist, and Robert Spurr Weston.

[30] Cons. Engr., Essen, Germany.

[30a] Received by the Secretary March 22, 1938.

[31] "Impounding Reservoirs as a Substitute for Biological Sewage Treatment Works in the Ruhr District," by Karl Imhoff, *Sewage Works Journal*, January, 1931.

[32] "Measures of Natural Oxidation in Polluted Streams," by H. W. Streeter, M. Am. Soc. C. E., *Sewage Works Journal*, March and May, 1935.

(4) Sunshine generates more oxygen from green water plants than the water will absorb from the surface.

(5) A completely still water surface takes little oxygen from the air. Every wave, whether from wind or boat, increases the oxygen absorption greatly.

(6) Sludge in sewage lagoons should be eliminated as much as possible by the use of settling tanks. Some bottom sludge will accumulate anyhow, because settling occurs in quiescent lagoons, even with pre-settled sewage. The accumulation of bottom sludge is continuous throughout the year, and is due to putrefactive bacteria which, in turn, use oxygen. The greatest demand on oxygen is during the warm season, just when the flow is diluted the least. The oxygen demand of the accumulated bottom sludge is at such times generally much higher than that of the dissolved organic matter which flows through uniformly. (The three sludge lagoons in the Ruhr Valley, Germany, have the advantage of being scoured by flood waters almost every spring, and even during the summer, they are not heavily loaded with sludge. If such natural washing does not occur, it may be advantageous to dredge the bottom.)

HENRY D. JOHNSON, JR.,[33] ESQ. (by letter).[33a]—The subject of planning for pollution control at Pittsburgh has been covered in a most able manner by Mr. Davis. The writer knows of no one in that city who is more familiar with this subject than he. It seems almost obvious that Pittsburgh engineers and municipal authorities must start entertaining plans for the treatment of the sewage and industrial wastes, but whether or not the time has yet arrived for building an extensive and expensive sewage treatment system is difficult to say.

The writer is personally interested in pollution control at Pittsburgh because of his interest in the possible recreational facilities afforded by the near-by streams. Much of the enjoyment of water sports is taken away by the sewage and industrial waste pollution which exists throughout the stretches of the three rivers within Allegheny County. When one is able to cruise up stream beyond the locations of large cities or boroughs, or above large industries and above those streams which drain mines, one finds the rivers in almost their primitive state. They are beautiful and attractive beyond words. It is not to be expected that any feasible plan of sewage treatment and industrial waste control will restore the rivers to their original purity, but there is no question that the pollution could be reduced sufficiently to make safe the use of the rivers for sports.

Mr. Davis' conclusion that primary treatment by sedimentation will be the only form of treatment required for some time to come, is in general correct, although certain communities that discharge their wastes into small streams will unquestionably have to consider complete sewage treatment in the immediate future.

Recent studies on the costs of the proposed plants indicate that it would be possible to afford very definite relief at a more or less nominal figure. Earlier

[33] Former Chf. Engr., Dept. of Public Works, Pittsburgh, Pa.

[33a] Received by the Secretary February 28, 1938.

reports and studies have been based on the premise that the effluent from treatment plants should be bacteriologically pure. However, a comparison of the costs of establishing water purification plants for down-stream communities, with the additional cost of sewage treatment plants which would provide a pure effluent, shows that, at present, it would be more economical to provide adequate water purification plants and only primary treatment for Pittsburgh and its environs. The primary treatment of sewage would unquestionably benefit those sections of the community which border the streams, because of the consequent elimination of sludge beds with their attendant foul odors and unsanitary conditions.

As Mr. Davis points out, the topography of the Pittsburgh District is such that consideration cannot be given to sewage treatment for the municipality of Pittsburgh alone; all or most of the cities, boroughs, and townships of Allegheny County must be considered. There are 116 municipalities using the three canalized rivers for disposal, and it would be futile for any one of these municipalities to attempt a solution of the problem without the co-operation of all the others.

To avoid petty politics and party dickering the only possible way to secure action is by legislative fiat. The time is now ripe for civic bodies and various leagues of voters to support legislation leading to the desired end.

The writer is not prepared to state that the plan for sewage treatment developed by the City of Pittsburgh is the best possible one. In any event, however, it can be made the basis of a final plan, and, as such, it should be endorsed by civic bodies for the purpose of obtaining the support of the citizens. Then, when the appropriate time comes, some plan (perhaps the present one) should be endorsed by the Legislature and each municipality should be required to adhere thereto. In lieu of the foregoing it might be proper for the Legislature to establish a Metropolitan District Plan, combining the municipal bodies of Allegheny County into a Sanitary District similar to that in Chicago, and empowering the officials of the district to raise funds to carry out and supervise the construction, maintenance, and operation of treatment plants, intercepting sewers, and, eventually, of disposal plants.

CHARLES M. REPPERT,[34] M. AM. SOC. C. E. (by letter).[34a]—The paper by Mr. Davis is a most interesting and timely discussion of one of the most important problems confronting Pittsburgh and its metropolitan area. There are some very unusual features involved, which must be fully taken into account. Inter-relationships with the acid mine drainage and proposed storage reservoirs are involved and there are many difficulties due to the nature of the topography. It is clear, as Mr. Davis states, that the situation can be met only by co-operative action of the many municipalities involved in adopting, financing, and carrying out a plan. The questions involved are: (1) The present necessity for reducing the sanitary load on the Ohio River in the interest of down-river water supplies, or for the prevention of general nuisance in the stream below Pittsburgh; (2) the need to prevent local nuisance

[34] Pittsburgh, Pa.

[34a] Received by the Secretary February 8, 1938.

along the shores of the rivers, particularly along the margin of the down-town business district; (3) the effect of reducing the acidity of the rivers by mine sealing; and (4) the effect of the construction of the system of proposed storage reservoirs for flood prevention and their utilization for increasing the dry-weather flow.

With regard to general nuisance at low stages in the Ohio River, below Pittsburgh, it can be stated that although a critical condition has been approached it has not been reached, probably because the increased sanitary load has been offset by the increased acid content of the rivers. It is evident that the effectiveness of mine sealing in reducing the discharge of acid mine water becomes one of extreme importance. It must be realized, however, that the acidity problem is one of control rather than elimination. Although the Monongahela River has been acid for many years, the Allegheny was alkaline until 1923; since that date it has shown a marked upward trend in acid content and in the frequency and duration of acid periods. As a matter of water conservation, the mine-sealing projects which have been carried out have been fully warranted in the public interest. On the other hand, a material reduction in the acid content during low-water periods would undoubtedly lead to a most aggravated condition of gross nuisance in the Ohio River below Pittsburgh. There is, however, a limit to the acid-reduction that can be secured by mine sealing, and it has not been demonstrated that the reduction will reach a degree that will affect the sewage disposal problem to a great extent. However, such possibility should be thoroughly investigated.

The sealing of the mines will be well worth while if the upward trend of concentration can be stopped and the extreme intensities prevented. It is evident that this particular question should receive further study, as it has great effect upon what provision should now be made for secondary treatment. If mine-sealing operations can be continued so as fully to cover the tributary coal areas, and if their effectiveness is such as to assure a very material reduction in acid content of the river water, then provision should undoubtedly be made for the early installation of secondary treatment works.

The effect of sewage discharge from the Pittsburgh area on down-river water supplies is of the greatest importance in determining the immediate necessity for sewage treatment at Pittsburgh. At the present time it appears that this effect is of concern during rising, rather than at low, river stages, and that critical conditions obtain or are approached during rises in the Ohio River, following prolonged low-water stages, at which times the sewage sludge deposited in the beds of the rivers at Pittsburgh and held in a state of retarded decomposition is scoured out and carried down stream. It is evident that, if the pollution contributed by Pittsburgh at the present time is materially affecting the difficulty and safety of purifying down-stream water supplies, any appreciable increase in the sewage load or the occurrence of abnormally long dry-weather periods would greatly aggravate the situation. Further data, however, should be made available as to the proportion of the pollution load contributed by Pittsburgh compared with that contributed from other and local sources.

What will be the effect of the construction of the system of proposed storage reservoirs now receiving the attention of local, State, and Federal authorities? Engineers of the Pittsburgh Flood Commission have estimated that the utilization of storage during dry-weather periods will increase the dry-weather flow in the Ohio River, at Pittsburgh, about 250 per cent. If this condition is brought about, then the necessity for secondary sewage treatment will be deferred for some years, and the number of primary treatment plants needed at present might be reduced. This would enable the adoption of a construction program covering a period of years.

The existence of local nuisance along the river shores at Pittsburgh constitutes one of the strongest local arguments for undertaking primary sewage treatment without delay. Such nuisance has increased in recent years, and has become particularly offensive on the margins of the down-town business district. Public decency and civic pride demand its abatement; if it is allowed to continue and to increase, depreciation of property values will result.

The writer is fully in accord with the proposed general plan described by Mr. Davis for the installation of a number of treatment plants located with due regard to economy and topography. Primary treatment will unquestionably suffice for a number of years, but it will be necessary to locate and design the treatment works in such manner that they will fit into the future installation of secondary plants. Likewise, the areas of the sites selected should be adequate for such future needs.

The first steps have been taken in solving the problem of pollution control at Pittsburgh. The Metropolitan Main Drainage Survey Project has made available necessary sanitary data and general plans and estimates. The definition of sanitary policy for this District has been made possible. Further studies should now be made of sewage characteristics and flow, acidity, and the effect of storage reservoir construction. The selection, availability, and cost of plant sites should be investigated in detail, with due reference to the cost of collection, which is an important matter because of topography and shore conditions. Finally, a decision should be made as to the program of construction—which, for financial reasons, should preferably be spread over a term of years—and the preparation of construction plans for the works to be first undertaken should be started so as to develop fully and settle all questions as to location, economy, and cost. These steps should be taken by the local authorities who will be responsible for the construction and operation costs of the works, in order that public funds be expended only to the extent that conditions require and with proper relation to other needs.

EDWIN K. MORSE,[35] M. AM. SOC. C. E. (by letter).[35a]—The paper by Mr. Ryder is unusually presented and very fair, and the writer has only one comment to make on it. Most of the head-waters of the Pymatuning are swamp lands and very flat country, which, of course, make the run-off of that river much more uniform than that of any other river west of the Allegheny Mountains in the Commonwealth of Pennsylvania. To the extent that this condition

[35] Cons. Engr., Pittsburgh, Pa.

[35a] Received by the Secretary February 4, 1938.

affects the results of the reservoir operation, the paper might be misleading to an engineer not acquainted with the location.

In several public talks on the Pymatuning, the writer has emphasized the necessity of building a reservoir on the Cussawago River, in a water-shed six miles from the Pymatuning Reservoir, and connecting it by a tunnel and open ditch to the latter. Then, when water is required from the Pymatuning Reservoir in the low-water period the gates of the Cussawago could be opened and the water of that reservoir be permitted to flow through the Pymatuning, thereby maintaining the lake level in the latter during that part of the season when it would be a great recreation center for approximately 3 000 000 people within a radius of seventy-five miles.

The dry season in this vicinity generally takes place from about the first of July to the first of November, but the schools open early in September, and the resort would be materially vacated by the first of October.

James H. Le Van,[36] Assoc. M. Am. Soc. C. E. (by letter).[36a]—General methods and laboratory tests applicable to a pollution survey of any stream are given in this paper, and Mr. Streeter has mentioned the fact that additional tests will depend upon the circumstances of the particular survey being made. It is very necessary that consideration be given to the uses that are made of the stream and to the presence of undesirable substances in the water, particularly to the substances originating in industrial wastes. A survey being made must take into account all pollution factors of the river under consideration. Too often only dissolved oxygen (D.O.) and bio-chemical oxygen demand (B.O.D.) tests are considered. The importance of the bacteriological and biological tests is not always realized. The former give the most sensitive indices of significant changes in the pollution of a river. The latter tests will "go hand in hand" with the other tests and will aid in solving questions that arise when unusual chemical and bacteriological results are obtained.

The U. S. Public Health Service is making a study of the Scioto River, from Columbus, Ohio, to its point of discharge into the Ohio River at Portsmouth, Ohio. The purpose of this survey is to observe the changes that occur in a polluted stream as the major sources of pollution are controlled by the construction of waste treatment plants. Already the City of Chillicothe, Ohio, has placed a new sewage treatment plant in operation. The City of Columbus closed its overloaded plant in 1937 and has turned all its sewage into the primary settling tanks of its large new plant, pending the placing of this activated sludge treatment plant in operation. The City of Circleville, Ohio, midway between Columbus and Chillicothe, is about to construct a plant. At this point wastes from a strawboard paper mill and several canneries enter the Scioto River. There are two paper mills at Chillicothe with wastes that discharge untreated into Paint Creek, one of the main tributaries of the Scioto River.

For this river study, it was necessary to select sampling points that would give information above and below the major sources of pollution, and above

[36] Passed Asst. San. Engr., U. S. Public Health Service; Officer in Chg., Scioto River Investigation, Chillicothe, Ohio.

[36a] Received by the Secretary February 26, 1938.

and below the river's main tributaries. Fortunately, highway and railroad bridges are situated at points where sampling stations should be established to obtain this information. As the survey progressed the results obtained showed the section where most rapid self-purification was taking place, and, consequently, where there should be the most frequent and intensive sampling. Likewise, stretches were revealed, where the river was becoming stabilized and where sampling need not be as frequent. As there is a limit to the laboratory work that can be performed by the personnel of a field laboratory, it is necessary that the officer in charge of it observe carefully to see where efforts should be concentrated and where unnecessary sampling may be discontinued. He must be on the alert for all changes that occur in the river and alter the work of the laboratory accordingly.

The following determinations are made on river samples at the Scioto River Investigation Laboratory, at Chillicothe: Temperature (at the time each sample is collected); suspended solids; methyl orange alkalinity; pH; dissolved oxygen (D.O.); 5-day bio-chemical oxygen demand (B.O.D.); bacteria growing on agar at 37°C, in 24 hr; and the most probable numbers (M.P.N.) of sewage bacteria (coliform group). In warm weather the dissolved oxygen and the bacteriological samples are transported by automobile to the laboratory in an iced container. Although the dissolved oxygen determination should be made as soon as possible after the sample is collected, the method used in this survey permits it to be done in the laboratory by a sanitary chemist instead of by a sample collector in the field. In addition, on one sample daily, B.O.D. determinations are made on identical samples with incubation periods of 3, 5, 7, 10, 12, 15, 20, and 25 days to check the course of the oxidation curve through the primary (carbonaceous) and secondary (nitrification) phases. Furthermore, one sample daily receives bacteriological examination through the completed test; the others receive the partly confirmed test using brilliant green bile (2%) broth. To save work in the field laboratory, composite samples preserved with concentrated sulfuric acid are sent monthly to the Stream Pollution Investigations' Laboratory, at Cincinnati, for nitrite and nitrate determinations. At times, these determinations are also made at the field laboratory to check up on advanced nitrification.

As part of the study of the Scioto River, a biologist makes weekly examinations of river samples. One week he examines freshly collected samples and the next week formaldehyde-preserved samples are sent to him at his Cincinnati headquarters. Mud samples are sent monthly to Cincinnati for examination. The biological forms play an essential part in the natural purification of a stream, and only by regular biological examinations can the changes in kind and relative numbers be recorded during and following a stream sanitation program. The biological examinations should never be slighted or minimized.

Unless the officer in charge has good reasons for doing so, all chemical, bacteriological, and biological examinations made on water samples should conform with the regularly accepted standard methods so that the results obtained will be comparable.

Present-day methods of automobile transportation permit samples to be transported to the laboratory with dispatch. It is more difficult to collect the samples at the correct stream points so that a representative sample will be secured each time the sampling can is lowered into the river. If bridges are not convenient, boats will be necessary unless the stream is so shallow that it can be waded. (The U. S. Public Health Service has a standard sample collecting can.[37]) Portable laboratories in automobile trailers can be used for the study of industrial wastes and untreated sewages when a large river system is surveyed.

When the study of stream pollution is undertaken nationally surveys will be made on many streams for which hydrometric data will not be available. Such situations will require the establishment of gaging stations on the main streams studied and on all their important tributaries, the regular collection of gage height records, and discharge measurements to prepare rating curves. Where such data cannot be obtained float measurements may have to be substituted, or some other method will have to be devised to obtain these data. It is necessary to know the different quantities of dilution water that enter a river system and the mean velocities and times of flow at different stages.

John C. H. Lee,[38] M. Am. Soc. C. E. (by letter).[38a]—It is axiomatic that, with public opinion in support of a law, much can be accomplished; but without such support, results may be incommensurate with effort. For years, and until recently, the public's attitude toward stream-pollution prohibition has been apathetic, and even antagonistic; but, of late, the Army Engineer has been able to quote Section 13 of the 1899 River and Harbor Act, and the 1924 Oil Pollution Act, effectively.

The Army Engineers have approached the problem along the Lower Delaware River enlisting the co-operation of industry, seeking a solution to each plant's problem that will be the least costly—if indeed an actual profit cannot be achieved from the by-products. The Army Engineers' inspectors receive welcome entrée to all plants, because the executives appreciate this attitude.

Publicity is probably as great a threat as prosecution. Industries spending thousands of dollars for public good-will are quick to realize the impending loss of that good-will, if they are posted as underminers of public health. It is not difficult to get plant executives to write their own "sentences"—what they will do to abate or remove pollution, and when—through co-operation rather than prosecution.

All that is required is patience and persistence and some persuasion. Army Engineers have contacted nearly two hundred industrial plants along the shores of Delaware River and Bay within the two years, 1936 and 1937. Without exception, all executives have responded—some more effectively, but all willingly. Impartial observers have expressed realization of improved conditions on the waters of the port, which are being sampled periodically.

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[37] *Public Health Bulletin No. 171;* also, Supplement No. 90, Public Health Repts., U. S. Public Health Service, Washington, D. C.

[38] Lt.-Col., Corps of Engrs., U. S. Army; with Office of Div. Engr., North Pacific Div., Portland, Ore.

[38a] Received by the Secretary March 9, 1938.

In order to bring the municipalities into effective co-operation, the Army Engineers, with the written approval of the local authorities, have written into a new project report for the Delaware's channel—Philadelphia to the Sea—the requirement that annual maintenance dredging of 100 000 and 10 000 cu yd must be performed, respectively, by the Cities of Philadelphia and Camden, N. J., until they shall have made effective installations of primary sewage treatment works.

The plan for the culm-polluted Schuylkill River is based upon the principle of co-operation between the anthracite industry and the public agencies charged with the administration of the plan of recovery and improvement.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

PRELIMINARY DESIGN OF SUSPENSION BRIDGES

Discussion

BY MESSRS. A. W. FISCHER, AND JACOB KAROL

A. W. FISCHER,[11] ESQ. (by letter).[11a]—In their "Introduction" the authors state that the so-called "elastic theory" used in earlier days is obsolete. The writer thinks, however, that for short spans of, say, 750 ft, or less, the "elastic theory" can be used. (Generally, for spans of 750 ft and less, other types of bridges are more economical.) Referring to the "deflection theory," the authors state that it is quite long and that a computing machine or a table of logarithms must be used in order to avoid possibilities of serious error. The writer agrees that the deflection theory is rather long and he would not use it, for that reason especially, for calculating the value of H_a. After H_a is known, however, the equations for moment and shear given by the "deflection theory," derived by using hyperbolic functions, are as short as any of the exact methods.

The authors state that the "trigonometric series" is not satisfactory for studies of a variety of preliminary designs. The writer much prefers the trigonometric series for the solution of H_a because a slide-rule can be used which gives results that are accurate enough for practical purposes. If the effect of the change of curvature is neglected, then the value of H_a can be calculated in a very short time, and the results will be amply accurate for preliminary design.

The authors then advance their method as being more rapid than the deflection theory or the trigonometric series method. After studying the proposed new method the writer is not convinced that any time is saved by it. It is true that the moments at the quarter-points and the center points of the main span and the moment at the center of the side span agree remarkably with the deflection theory, but even so the method requires considerable computation. The writer did not check the moment of 113 300 ft-kips as given for the quarter-point by the authors' preliminary design for the Manhattan Bridge, but

NOTE.—The paper by Shortridge Hardesty and Harold E. Wessman, Members, Am. Soc. C. E., was published in January, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[11] Care, Constr. Service, Veterans Administration, Washington, D. C.

[11a] Received by the Secretary March 8, 1938.

it seems that the results are somewhat too small. By solving for H_a, using the trigonometric series,[12] with the live load covering 0.425 of the main span, neglecting the effect of the change of curvature, and assuming that the ratio of H_a to H_w is 0.2, the writer obtains an answer of 2 028 kips for H_a, and substituting this in an equation involving hyperbolic functions,[13] the value of the moment at the quarter-point is + 118 870 ft-kips. If the effect of the change of curvature is considered, H_a = 2 040 kips and the moment at the quarter-point is + 118 650 ft-kips.

For the Manhattan Bridge, by solving for H_a, using the trigonometric series with the live load covering 0.3 to 0.7 of the main span, considering the effect of the change of curvature, and assuming that the ratio, H_a to H_w, is 0.3, the writer finds 3 166 kips for H_a; and substituting this value in an equation involving hyperbolic functions the value of the moment at the center of the main span is + 88 300 ft-kips. In their preliminary design the authors give 91 300 ft-kips.

In their entire paper the authors do not mention how they propose to compute shear, and for a preliminary design it is also important that the shear is known. The value of H_a can also be readily solved by the trigonometric series, and then substituting in a general equation containing hyperbolic functions,[14] the value of shear can be ascertained.

For a preliminary design for moment at the quarter-points and the center of the main span, etc., it also seems to the writer that such a method as given by Arvid H. Baker, Assoc. M. Am. Soc. C. E.,[15] is considerably shorter than the authors' method, and the results are such that, for a preliminary design, they are satisfactory. The moment is first solved by reading coefficients from a chart introduced by D. B. Steinman, M. Am. Soc. C. E.,[16] and then applying a correction factor derived by Mr. Baker.[15]

Throughout the entire paper, the writer fails to grasp the meaning of just what is meant by "preliminary design of stiffening trusses of suspension bridges." If it means a preliminary design for estimating purposes, it seems to the writer that too much time will be required when compared with the chart presented in 1926 by the late J. A. L. Waddell, Hon. M. Am. Soc. C. E.,[17] from which the steel in the stiffening trusses must be read by an experienced engineer familiar with suspension bridge designs. Any one who knows the cost of steel in place can make a very close estimate in a short time, and for that reason the writer really sees no particular need for a preliminary design such as that given in the paper.

In their closing discussion the authors should show just how they propose to apply their method to the computation of shear in the main span and in the side spans and, furthermore, they should present a general method for calculating the moments at points other than the quarter-points and center.

[12] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 387.

[13] *Loc. cit.*, Vol. 100 (1935), p. 1199, Equation (80) ; see, also, p. 1200.

[14] *Loc. cit.*, Equation (81) ; and, also, p. 1200.

[15] "Suspension Bridge Analysis by the Exact Method Simplified by Knowledge of Its Relations to the Approximate Method," by Arvid H. Baker, Eng. and Science Series No. 24, Pl. I and II, Rensselaer Polytechnic Inst., June, 1928, Troy, N. Y.

[16] "A Practical Treatise on Suspension Bridges," by D. B. Steinman, Charts I, II, and III.

[17] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 894, Fig. 5; see, also, p. 942, Table 5.

JACOB KAROL,[18] ESQ. (by letter).[18a]—The principal merit of this valuable paper lies in the "picture" it presents of the related action of the cable and the stiffening truss. The writer wishes to discuss this pictorial aspect in greater detail.

It has been emphasized, in the literature on the deflection theory of suspension bridges, that stresses for a combined loading cannot be found by a summation of the stresses due to the component loads. Although this statement is strictly true, it is also true, and much more important, to state that superposition of component effects under certain conditions gives a result that is approximately correct.

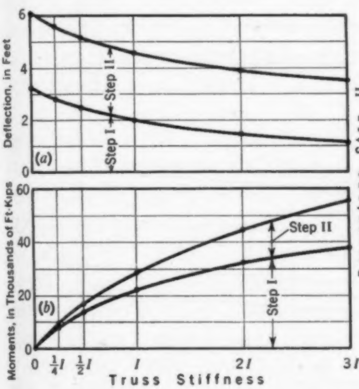

FIG. 8.—EFFECTS AT QUARTER-POINT OF MAIN SPAN OF TRIBOROUGH BRIDGE DUE TO CHANGES IN TRUSS STIFFNESS (LOADING CONDITION, DEAD LOAD + LIVE LOAD + 55° CHANGE OF TEMPERATURE)

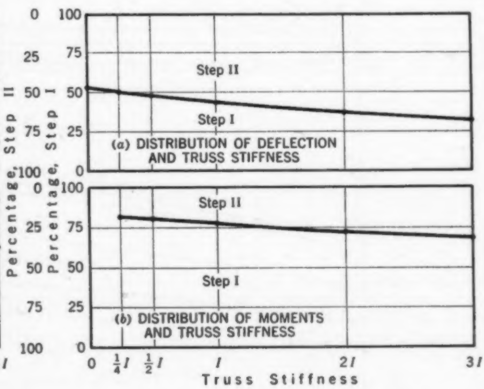

FIG. 9.—EFFECTS AT QUARTER-POINT OF MAIN SPAN OF TRIBOROUGH BRIDGE DUE TO CHANGES IN TRUSS STIFFNESS (LOADING CONDITION, DEAD LOAD + LIVE LOAD + 55° TEMPERATURE)

The authors' method of calculation permits a direct determination of these component effects. In Professor Wessman's original thesis[5] the effect due to the addition of the stiffening truss to the unstiffened cable was designated as Step I, and the combined effect of cable stretch, temperature change, and side-span interaction was designated as Step II. It may be noted, in passing, that Equation (14*b*), which gives the moment due to Step I, is based on Equation (13), which is an assumption. The writer found that the moment due to Step I, as calculated by the deflection theory, was slightly less than that calculated by the authors' method, whereas the reverse was true of the deflection. However, the effect of Step II as computed by the authors' method is compensating, so that the final moment and deflection are in substantial agreement with the values obtained by the deflection theory.

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[18] Graduate Fellow in Civ. Eng., Univ. of Illinois, Urbana, Ill.

[18a] Received by the Secretary March 12, 1938.

[5] "Related Functions of Cable and Stiffening Truss in Suspension Bridges," by Harold E. Wessman, M. Am. Soc. C. E., presented to the Univ. of Illinois in 1936, in partial fulfillment of the degree of Doctor of Philosophy in Engineering.

From the preceding discussion it is obvious that the separation of the total moment and deflection into their component effects is quantitatively approximate. It may be added that this procedure is applicable to the main span only.

Economical design implies an understanding of the relations among the various factors entering into the design. Hence, only by studying the changes produced by the variation of the several factors can one obtain the necessary understanding.

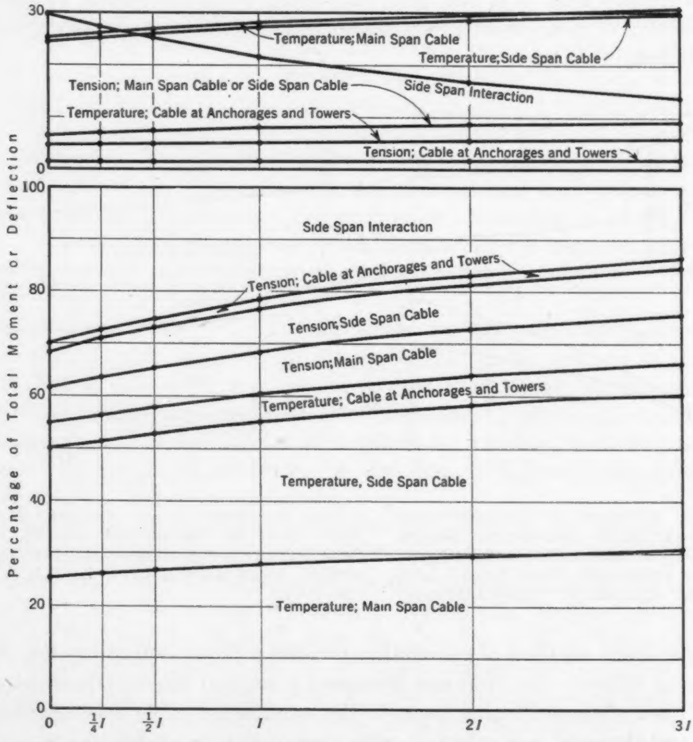

FIG. 10.—EFFECTS AT QUARTER-POINT OF MAIN SPAN OF TRIBOROUGH BRIDGE ON MOMENTS AND DEFLECTIONS FROM CABLE SAG, DUE TO CHANGES IN TRUSS STIFFNESS (LOADING CONDITION, DEAD LOAD + LIVE LOAD + 55° TEMPERATURE)

Consider, as an illustration, the positive moment and deflection at the quarter-point of the main span of the Triborough Suspension Bridge, in New York, N. Y., as affected by a variation in truss stiffness, with the ratio of the main span truss stiffness to that of the side span held constant. Fig. 8(*a*) indicates the relation between the deflection and the truss stiffness, and Fig. 8(*b*), the relation between the moment and the truss stiffness. These same data may be plotted in another manner as proportions of the total moment or deflection. Fig. 9(*a*) indicates that the proportion of the total deflection due to Step I is 53% for the unstiffened cable and 31% for a truss stiffness of 3 *I*

(I is the truss stiffness as designed), and Fig. 9(b) shows that the proportion of the total moment due to Step I is 82% for 0.25 I and 68% for 3 I.

Fig. 10 indicates the proportions of the total moment or deflection from Step II, due to its component factors, in relation to the truss stiffness. It shows that the effects of cable stretch due to temperature and tension are sensibly constant, and that side-span interaction causes 30% of the total for an unstiffened cable and 14% of the total for a truss stiffness of 3 I. By considering both Figs. 9 and 10, it seems that about 50% of the total deflection and about 20% of the total moment is due to the unvarying cable stretch effects of Step II.

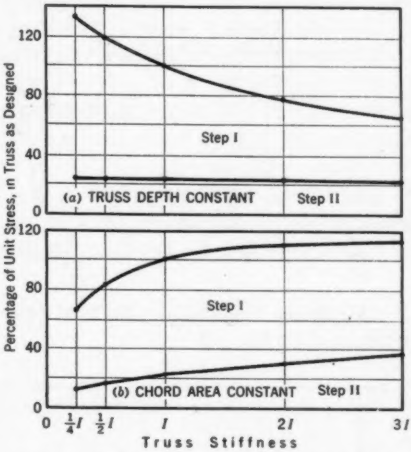

FIG. 11.—EFFECTS AT QUARTER-POINTS OF MAIN SPAN OF TRIBOROUGH BRIDGE ON UNIT STRESSES IN STIFFENING TRUSS DUE TO CHANGES IN TRUSS STIFFNESS (LOADING CONDITION, DEAD LOAD + LIVE LOAD + 55° TEMPERATURE)

Another interesting study is the effect of the truss proportions on the stresses in the chords. Assuming that the truss as designed has a stress of 100%, the preceding data were plotted in Fig. 11. For a constant truss depth, Fig. 11(a) indicates that the stress due to Step II is approximately constant, whereas that from Step I decreases with an increase in I. Fig. 11(b) shows that for a constant chord area the stresses due to Step I and Step II both increase with an increase in I. Hence, if a structure has been designed with a truss of stiffness, I, depth, d, and chord area, A, the following relations obtain:

Decreasing the value of I, with depth, d, constant, increases the unit stress.
Increasing the value of I, with depth, d, constant, decreases the unit stress.
Decreasing the value of I, with Chord A constant, decreases the unit stress.
Increasing the value of I, with Chord A constant, increases the unit stress.

If it is desired to maintain a constant unit stress, the required relationships can be determined as follows: For the designed truss, let I represent the stiffness; M, the moment; A, the chord area; d, the depth; and s, the unit stress. For a truss of stiffness, $I_n = n\,I$, the corresponding terms are denoted with a subscript, n. Letting $I = \frac{A\,d^2}{2}$,

$$s = \frac{M\,d}{\frac{2\,A\,d^2}{2}} = \frac{M}{A\,d} \qquad (43)$$

Similarly,

$$s_n = \frac{M_n}{A_n\,d_n} \qquad (44)$$

From Equations (43) and (44),

$$\frac{M}{A\,d} = \frac{M_n}{A_n\,d_n} \qquad (45)$$

Also, by definition:

$$A\,d^2 = \frac{1}{n} A_n\,d_n^2 \qquad (46)$$

From Equations (45) and (46),

$$\frac{A_n}{A} = n\left(\frac{d}{d_n}\right)^2 = \frac{M_n}{M} \times \frac{d}{d_n} \qquad (47)$$

or,

$$\frac{d_n}{d} = n\,\frac{M}{M_n} \qquad (48)$$

and,

$$\frac{A_n}{A} = \frac{1}{n}\left(\frac{M_n}{M}\right)^2 \qquad (49)$$

Applying Equations (48) and (49) to the preceding data gives the results indicated in Fig. 12.

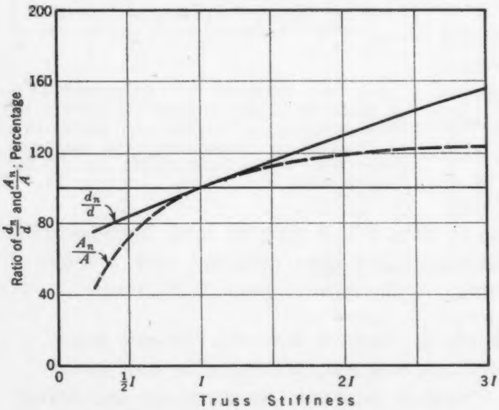

FIG. 12.—EFFECT AT QUARTER-POINT OF MAIN SPAN OF TRIBOROUGH BRIDGE; REQUIRED VARIATION IN TRUSS DEPTH AND CHORD AREA FOR CONSTANT UNIT STRESS WITH VARYING TRUSS STIFFNESS

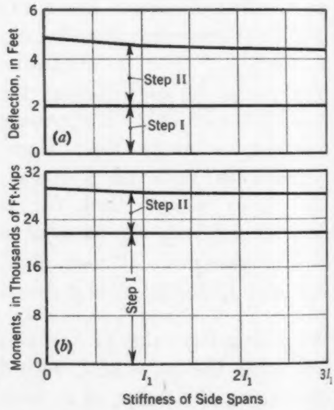

FIG. 13.—TRIBOROUGH BRIDGE; MAXIMUM EFFECTS AT QUARTER-POINT OF MAIN SPAN FOR DESIGN STIFFNESS I IN MAIN SPAN AND VARIOUS VALUES OF I IN SIDE SPANS

The writer made some further studies to determine the effect of the side-span stiffness on the positive moment and deflection at the quarter-point of the main span, the stiffness of the latter being kept constant and equal to the design value. Fig. 13 indicates a reduction of about 4% in the moment and 10% in the deflection for the extreme variation in side-span stiffness from 0 to $3\,I_1$. This, and other, studies by the writer seem to indicate that for a given cable and span layout, the moments and deflections in any span are primarily a function of the stiffness of that span. This statement does not apply unreservedly to the case of continuous stiffening trusses. It should be evident from the fore-

going that the authors' method of analysis is of undoubted value in giving a clear and simple picture of the action of a suspension bridge with hinged stiffening trusses. This picture is admittedly approximate quantitatively, but not so approximate as to be misleading. The writer has extended the authors' method to include the determination of the maximum shears in the stiffening trusses. From 80% to 90% of the total shear is due to Step I.

In closing, the writer wishes to take exception to the authors' criticism of the deflection theory on the ground that it requires long and involved computations. He has developed an influence line procedure for the generalized deflection theory whereby slide-rule computations are accurate enough for all practical purposes.

Acknowledgment.—These studies of suspension bridges were made under the direction first of Hardy Cross, and, later, of T. C. Shedd, Members, Am. Soc. C. E., as part of the writer's graduate study, and their helpful comments are hereby acknowledged.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

RELATIVE FLEXURE FACTORS FOR ANALYZING CONTINUOUS STRUCTURES

Discussion

By Messrs. George W. Housner, Homer M. Hadley, Adolphus Mitchell, John B. Wilbur, and Leon Blog

George W. Housner,[15] Jun. Am. Soc. C. E. (by letter).[15a]—The method of presenting this paper can be criticized in that Mr. Stewart professes to derive his solution from the theory of moment-areas and from geometrical principles. Upon examination, however, it is seen that the method of traversing continuous structures is identical with the conjugate beam theory, except that the author chooses to represent the $\frac{M}{E\,I}$-areas graphically as angle changes.

Although the author sketches a traverse of the deflected structure he reasons from the conjugate beam theory. For example, under the heading "Application To A Continuous Beam of Non-Tapering Spans," he states "Now, assign arbitrary values of 2, 6, and 4, to the angles of the triangle in the traverse of the right span, these values being proportional to the opposite sides of the triangle." This statement is objectionable, because a triangle with sides proportional to 2, 6, and 4 is an impossibility. These proportions define a straight line and not a triangle. They are perfectly proper in any application of the conjugate beam theory as distinct from the principles of geometry.

To demonstrate that the method of traversing continuous structures is the same as the conjugate beam theory, the author's first example is reproduced in part. For any real beam, construct a fictitious beam of equal length loaded with the $\frac{M}{E\,I}$-diagram. Then the shear at any point on the fictitious beam is equal to the rotation at that point on the real beam, and the bending moment at any point on the fictitious beam is equal to the deflection at that point on

Note.—The paper by Ralph W. Stewart, M. Am. Soc. C. E., was published in January, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1938, by Messrs. Frederick Shapiro, and Dean F. Peterson, Jr.

[15] Pasadena, Calif.

[15a] Received by the Secretary February 15, 1938.

the real beam. Therefore, given the continuous beam shown in Fig. 9(*a*), the approximate bending moment diagram can be sketched as shown in Fig. 9(*b*).

Assigning an arbitrary value of 2 to the reaction at End *E*, it follows from statics that the triangular load on the fictitious beam, *D E*, equals 6 and the left reaction of Beam *D E* equals 4. These beam reactions represent the rotation of the real beam at the supports. Therefore, the right reaction of Span *C D* is also 4 since there is continuity over the supports. Then, since M_{de} equals M_{dc} the value of the triangle load (measured by M_{dc}) is $6 \times 1 \div 2 = 3$. The deflection at Reaction *C* is zero, so that, taking moments about Point *C*, with the length of beam equal to 3, Area $\times 1 - 3 \times 2 - 4 \times 3 = 0$ and the area equals 18. From statics the reaction at Point *C* equals $18 - 3 - 4$ which is 11.

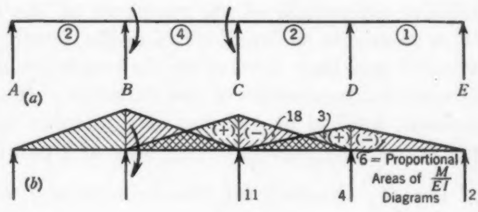

FIG. 9

This reasoning continued will reproduce the author's solution. The other examples in the paper can be solved similarly.

Although it may be simpler to sketch the traverse of a beam than to sketch the bending moment diagrams, the use of the traverse is less obvious. This is to be expected as it is a secondary form of the conjugate beam theory. In view of this it is doubtful whether the use of the traverse is justified as it somewhat obscures the reasoning, thus making the solution more difficult to comprehend.

HOMER M. HADLEY,[16] ASSOC. M. AM. SOC. C. E. (by letter).[16a]—When the tangled complexity of the Gordian knot was laid before Alexander that he should untie it, he drew his sword and cut it. This was a wholly new approach to the problem but it effectively disposed of difficulties.

Mr. Stewart has devised a somewhat similar method of dealing with stress distribution in continuous structures, at least in all their common varieties. Essentially, it is recognition of the fact that the flexed form of a beam is a direct expression of the stresses that are in it. However, his method proceeds a step further in recognizing that the direction of the tangents to the elastic curve at the ends of a member and the direction of an intermediate course connecting critical points upon the end tangents tell all that needs to be known —the direction and magnitude of the end moments. Therefore, in lieu of much involved clutter there is presented a method of analysis which, by means of the most elementary geometry and a fundamental or two of mechanics, yields an exact solution by a single operation.

It is not the least important part of Mr. Stewart's method that its first step is to draw a traverse diagram, the shape of which reflects the deformations in the structure it represents. This diagram requires no scale, and it can be

[16] Regional Structural Engr., Portland Cement Assoc., Seattle, Wash.

[16a] Received by the Secretary February 21, 1938.

exaggerated and distorted so long as it shows, correctly, the inclination of its courses with respect to one another. It is a framework upon which the relative angular changes are calculated and recorded. It furnishes, at all times, a valuable image of the structure under consideration, its various inclined courses indicating clearly the deformations and stress tendencies in the original subject.

However, the essential feature of the method is its determination of the relative magnitude of the moments at the ends of a member by expressing these moments as functions of angles developed at certain critical points in a traverse and then developing the respective magnitudes of the angles from the geometrical necessities of the situation. It is because the flexural effects of a moment applied at one end of a member can be represented graphically by the proper intersection of two lines at a point offset from the center of gravity of the $\frac{M}{E\,I}$-diagram for this moment and member that this method has the elegant nicety which characterizes it.

Beginning at the member in the system most remote from the one subjected to applied stress, convenient relative values are assigned to the angles developed in it under the conditions of end restraint which prevail at its extremity. Thence, one proceeds to the next member and the relative values of the angles in its traverse are precisely determined from the relationship which that member's stiffness bears to the first member's stiffness; and so on, progressively obtaining angle values which are accurate and exact in their relationship to one another. Either a final or concomitant conversion of these angles gives a set of end moments likewise accurately related to one another and to the applied moment that produces them. With this series of moments established, multiplying it by the ratio which any other applied or causal moment bears to the applied moment of the established series gives correct values for the new series. It is simplicity itself.

The writer believes that Mr. Stewart could have improved his paper if, at the outset of his illustrative problems, he had stated in definite categorical detail the procedure to be followed: Do this—then do this—then do this, etc. After all, it is in the use and application of a method that its value resides. The reasons for the various steps can be elaborated in the discussion of the several examples.

Fig. 4(*b*) would be improved if "stiffnesses" were substituted for "Relative Δ" in each case. When every one is familiar with stiffnesses, why introduce another term for another aspect of the same thing?

ADOLPHUS MITCHELL,[17] JUN. AM. SOC. C. E. (by letter).[17a]—The writer has used the traverse method since its first presentation.[18] The ensuing years have given the opportunity to test the method in the design office and classroom. It is an attribute that engineers familiar with the area-moment method experience no difficulty in analyzing continuous structures by traversing the elastic

[17] Structural Designer, State Highway Dept., Santa Fé, N. Mex.

[17a] Received by the Secretary February 28, 1938.

[18] *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 105; its first publication, without discussion, was in *Proceedings*, Am. Soc. C. E., October, 1934, p. 1125.

curves. The present paper does much to simplify further the traverse method and is a distinct contribution to engineering literature.

The subject of closed box-frames has scarcely been touched. The true shape of the traverse is seldom known to the designer at the beginning of an

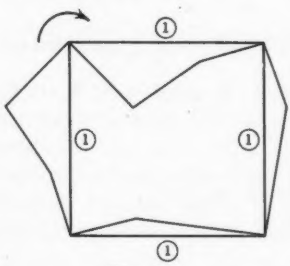

FIG. 10

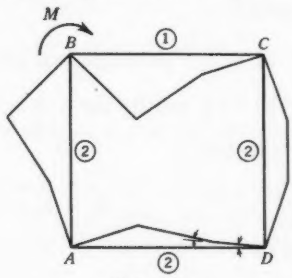

FIG. 11

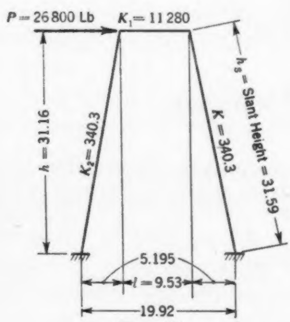

FIG. 12

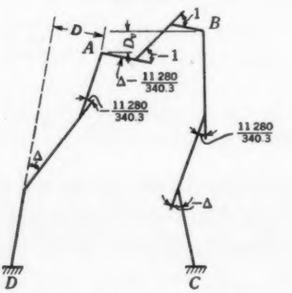

FIG. 13

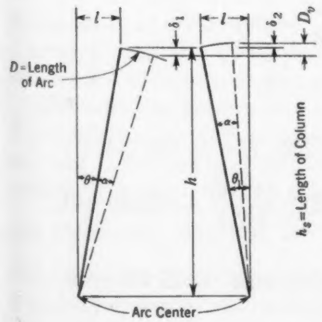

FIG. 14

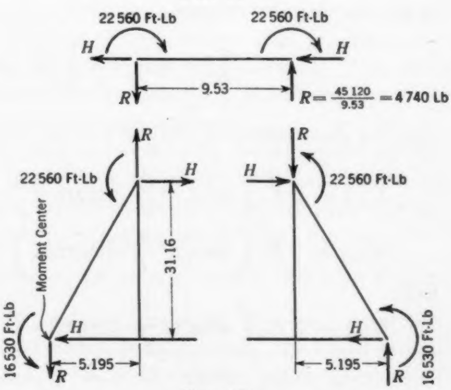

FIG. 15

analysis. This point is illustrated by Figs. 10 and 11 showing two variations of Fig. 5. The problem becomes more difficult in multiple box-frame design especially in extending existing culverts many of which utilized 12-in. interior walls. The writer would like to see a general solution of this difficulty that would require little added work for the designer.

Many problems show great advantages in the traverse solution. The following problem was solved by the slope-deflection method by Mr. A. W. Fischer, in 1936.[19] Fig. 12 shows the required dimensions and values of K, and Fig. 13 shows the traverse. To determine D_v, Fig. 14 will be used. Applying trigonometry,

$$\delta_1 = h - h_s \cos(\theta + \alpha) = h - h_s (\cos\theta \cos\alpha - \sin\theta \sin\alpha) \ldots\ldots (1)$$

Since α is very small, $\sin\alpha = \alpha$; $\cos\alpha = 1$; $h_s \cos\theta = h$; $h_s \sin\theta = l$, and $\alpha = \frac{D}{h_s}$. Hence,

$$\delta_1 = h - h + l\alpha = l\alpha = \frac{lD}{h_s} \ldots\ldots (2)$$

Likewise,

$$\delta_2 = h_s \cos(\theta - \alpha) - h = h_s (\cos\theta \cos\alpha + \sin\theta \sin\alpha) - h$$

$$= h + l\alpha - h = \frac{lD}{h_s} \ldots\ldots (3)$$

and,

$$D_v = \delta_1 + \delta_2 = 2\frac{lD}{h_s} = 0.329\, D \ldots\ldots (4)$$

Keeping in mind that breaks in the traverse occur at the third points of members of constant moment of inertia and using the traverse of the beam, the offsets at End B, Fig. 11, may be added to obtain,

$$0.329\, D + \left(\Delta - \frac{11\,280}{340.3}\right) 9.53 - \frac{2}{3}(9.53) + \frac{1}{3}(9.53) = 0 \ldots\ldots (5a)$$

Similarly, for the column,

$$\frac{2}{3}(31.59)\,\Delta - \frac{1}{3}(31.59)\,\frac{11\,280}{340.3} - D = 0 \ldots\ldots (5b)$$

Solving Equations (5) simultaneously, $\Delta = 26.3$; and:

$$M_{AB} = 2\,K_1 \left(\text{area of } \frac{M}{I}\text{-diagram}\right) = 2(11\,280)\,1 = 22\,560 \text{ ft-lb};$$

$$M_{AD} = 2\,K_2 \left(\text{area of } \frac{M}{I}\text{-diagram}\right) = 2(304.3)\,\frac{11\,280}{340.3} = 22\,560 \text{ ft-lb};$$

and,

$$M_{DA} = 2\,K_2 \left(\text{area of } \frac{M}{I}\text{-diagram}\right) = 2(340.3)(26.3) = 17\,930 \text{ ft-lb}.$$

These values are based on an assumed area of the $\frac{M_{AB}}{I}$-diagram equal to unity. The corresponding value of P is obtained by taking moments as shown in Fig. 15, giving $P = 2\,H = 4\,180$ ft-lb.

[19] "Successive Elimination of Unknowns in the Slope Deflection Method," by John B. Wilbur, Assoc. M. Am. Soc. C. E.; discussion by A. W. Fischer, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), Fig. 7, p. 362.

The given value of P is 26 800 lb and the correct moments are obtained by direct proportion, resulting in $M_{AB} = 22\,560\left(\frac{26\,800}{4\,180}\right) = 144\,600$ ft-lb; $M_{AD} = 22\,560\left(\frac{26\,800}{4\,180}\right) = 144\,600$ ft-lb; and,

$$M_{DA} = 17\,930\left(\frac{26\,800}{4\,180}\right) = 115\,000 \text{ ft-lb.}$$

All computations have been made with a slide-rule and the explanation assumes familiarity with the author's paper.

For continuous beams the three-moment theorem and moment distribution cannot compare with the author's method. This is especially true when members of variable cross-section are involved because of the directness of its application. Slopes and deflections at any point are easily found by using the load, shear, moment, slope, and deflection series of diagrams as repeatedly brought out in discussions of the shear-area method.[20] Simpson's rule may be used to advantage when great accuracy is desirable in handling areas bound by complex curves.

No method is best for handling all problems, but any designer would be well repaid for time spent in learning when to use the Stewart traverse method.

JOHN B. WILBUR,[21] ASSOC. M. AM. SOC. C. E. (by letter).[21a]—In presenting a new expedient for analyzing continuous structures, Mr. Stewart has contributed a paper of real interest. Just what arrangement of the details of a solution leads to the best results is a difficult matter to determine, but certainly Mr. Stewart's procedure for analyzing continuous structures has the merit of simplified computation under some circumstances.

In simplifying analysis procedure, there is always danger that the reasoning involved will become obscure. The writer is of the opinion that simplicity of thought is of at least equal importance with simplicity of procedure. With this in mind, it might be questioned as to whether the concept of flexure factors does not pay too great a price for the advantage it may gain.

Although the process of traversing the elastic curve by assuming the slopes at certain joints, and progressively computing slopes at adjacent joints, is excellent under some circumstances, the same results can be accomplished with precisely the same mathematical steps by working directly from the moment-area theorems. Treating the $\frac{M}{EI}$-curve as an elastic load, it may be shown easily that the following theorem holds: If a section, A–B, of a member which is initially straight, has its slope changed due to bending only, the slope at one end of the section, referred to the chord, A–B, of the elastic curve, is equal to the reaction at that end of an imaginary beam, A–B, which is carrying a distributed elastic load of intensity, $\frac{M}{E\,I}$.

[20] *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 947.

[21] Associate Prof. of Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

[21a] Received by the Secretary March 14, 1938.

For the case of an end-supported beam, if the entire beam is considered as the Section $A-B$, this theorem leads directly to the computation of true slopes referred to the original position of the axis of the beam, inasmuch as this original position of the axis coincides with the chord, $A-B$.

In applying this theorem, the following sign convention may be observed: (*a*) Positive values of $\frac{M}{E\,I}$ correspond to a downward elastic load; (*b*) negative values of $\frac{M}{E\,I}$ correspond to an upward elastic load; (*c*) an elastic reaction that acts in a direction to cause positive shear in the adjacent section of the beam considered, corresponds to a positive or clockwise slope; and (*d*) an elastic reaction that acts in a direction to cause negative shear in the adjacent section of the beam considered, corresponds to negative or counter-clockwise slope.

To determine the slopes obtained by Mr. Stewart in Fig. 2(*b*), by means of this theorem, one may proceed as follows:

(1) Sketch the resultant moment curve for the case under consideration as shown in Fig. 16(*a*), and sub-divide this moment curve into its component

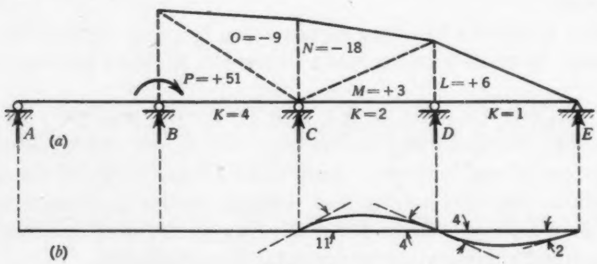

FIG. 16

parts as shown by dotted lines. Note that all moments may be assumed as positive.

(2) Assume that the slope at End E is -2. Note that this assumption corresponds to a counter-clockwise slope, as will actually occur at this point. It also corresponds to an upward reaction due to the $\frac{M}{E\,I}$ loading, since this produces negative shear to the left of End E.

(3) In order that the triangular elastic load shown by Area L may produce an upward reaction of 2 at End E, when acting on the end-supported beam, $D-E$, Area L must equal $+6$; the elastic load, $\frac{M}{E\,I}$, must act downward, corresponding to positive moment.

(4) The elastic load, $L = +6$, causes an upward reaction of 4 at Point D. This reaction causes positive shear to the right of Point D; and, the slope at this point, therefore, is equal to 4, and is clockwise.

(5) The moment area, L, is given by:

$$\frac{M_{DE}}{E\,I_{DE}} \times \frac{L_{DE}}{2} = \frac{1}{2\,E} \times \frac{M_{DE}}{K_{DE}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Similarly, the moment area, M, is given by:

$$M = \frac{1}{2\,E} \times \frac{M_{DC}}{K_{DC}} \quad \text{.......................} (7)$$

Since $M_{DE} = M_{DC}$, the following relation holds:

$$\frac{M}{L} = \frac{K_{DE}}{K_{DC}} \quad \text{.........................} (8)$$

For the case under consideration, $\frac{K_{DE}}{K_{DC}} = \frac{1}{2}$, so that $M = \frac{1}{2}(+6) = +3$.

(6) Clockwise slope at Point D corresponds to positive elastic shear to the left of D in the beam, $C\,D$; hence, for purposes of considering the equilibrium of the elastic loads on Beam $C\,D$, the elastic reaction at Point D is downward, and equals 4. This reaction is due to the elastic loads, N and M, acting on Beam $C\,D$; whence, $N \times 1 + (M = +3) \times 2 + 4 \times 3 = 0$; and $N = -18$. This negative value shows that the elastic load, N, acts upward, corresponding to negative moment.

(7) The slope at Point C is now obtained by evaluating the elastic reaction at that point. The upward value of this reaction equals $\frac{1}{3}(3) - \frac{2}{3}(18) = -11$; the reaction, therefore, is down; this causes negative shear to the right of Point C; and the slope at that point, therefore, is counter-clockwise.

In a similar manner, relative slopes may be determined at other joints. From the relative slopes, relative moments are easily determined.

Mr. Stewart refers to a solution for a continuous beam given by the writer[5] and suggests a comparison of the labor involved by the two methods. As explained in the closing discussion of the writer's paper, this example was included as a pedagogical step in leading the reader gradually into the more complicated problem of a building frame, rather than because it was believed to illustrate a desirable method of analyzing continuous beams.

Although the technique of Mr. Stewart's solution differs considerably from that suggested by the writer, the two methods have certain features in common. Both are based fundamentally on the moment-area theorems, and both work, successively, from joint to joint.

LEON BLOG,[22] Assoc. M. AM. Soc. C. E. (by letter).[22a]—Eight years have transpired since, in May, 1930, Hardy Cross, M. Am. Soc. C. E., published his valuable method of analysis.[23] As has been the case with the original analyses for continuous beams and for single span and continuous span arches, such publication has stimulated the presentation of improvements and of alternative methods of obtaining the same results as by the prototype. Of these additions to the subject, the one offered by T. Y. Lin, Jun. Am. Soc.

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[5] "Successive Elimination of Unknowns in the Slope Deflection Method," by John B. Wilbur, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 347, 356, and 1048.

[22] Asst. Chf. Structural Engr., Div. of Bridges and Structures, City of Los Angeles, Los Angeles, Calif.

[22a] Received by the Secretary March 22, 1938.

[23] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1.

C. E., in 1934,[24] was a variation of the Cross method. The writer has shown that the Lin method was applicable to one-story structures and offered quicker solutions than the Cross method when such structures had numerous spans. It was followed by one presented by the author[25] which was a combination of the area-moment method and a set of simultaneous equations for each member, based upon the concept that for every part of a bent beam a triangle may be conceived which has, for its three sides, the line joining the ends of the bent part and the two end tangents. The tangents form an angle which is equal to the sum of the two interior angles of the triangle and is measured by $\frac{M}{EI}$, which is Moment-Area Principle 1.[26] The series of tangents, when taken continuously along a beam or a structure, may be considered as a surveyor's traverse in conjunction with the attendant angles, a concept which gives Mr. Stewart's method its name.

The traverse method was followed by L. E. Grinter, Assoc. M. Am. Soc. C. E., in devising his method of balancing angle change,[27] and it, in turn, is now followed by a restatement of the Stewart traverse method which eliminates the cumbersome equations of the original presentation.

The purpose of this discussion is to register the advantages and disadvantages of each of these developments based upon the following criteria:

(1) Is the method a general tool, or is it a special tool of analysis?
(2) Must approximations be made as to the degree of end restraint in the terminal members in the highest story?
(3) Does the method yield approximate or accurate results?
(4) Is the method faster than its competitors?
(5) Is it easy to remember?

The Cross method is a general tool. It applies to all true continuous frames whether they are single or multi-storied. Distribution can be begun without approximations as to restraint in either the beams or the columns. This method yields approximate results which are accurate enough for the purpose. For certain problems it is not as fast as some of its competitors. It is the easiest method to remember.

The direct method of moment distribution, by Mr. Lin, is not a general tool. It is not applicable to multi-storied structures without approximation as to end restraint in the terminal members in the highest story. The method yields approximate results which are as accurate as the Cross results. For one-story structures (such as bridges) for which influence lines are required, it is faster than the Cross method despite the extra preliminary computations required for the modified stiffness and carry-over factors which lead to its rapid convergence. Like the Stewart method it is a one-direction method. It is not as easy to remember as the Cross method.

The Grinter method of analyzing continuous frames by balancing angle changes is a general tool. It is not moment distribution in the strict sense

[24] *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 61.
[25] *Loc. cit.*, Vol. 101 (1936), p. 105.
[26] "Mechanics of Engineering," by I. P. Church, pp. 490–492 incl.
[27] *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1020.

of the term, although some of the moment distribution technique, such as balancing around a joint and carrying over angles instead of moments, is used. Approximations at the upper stories need not be made. The answers are approximate, but accurate enough for most purposes. It is distinctly not faster than its competitors and is the most difficult of all the methods proposed after the Cross method to remember. It is really slope deflection in reverse.

The restatement of the Stewart method is a special tool. It is not applicable to multi-storied structures inasmuch as assumptions as to restraint of the terminal members of the highest story must be made. For one-story, or closed, frames it is the most accurate of all the methods under discussion because it is distinctly not a method of approximation. This is because no convergence is involved in the solution of problems. As a special tool, it is faster than all the methods being discussed. This statement is made with the reservation that the user is not under the necessity of familiarizing himself with the method every time he has to use it. It is of use to those who are continually analyzing one-story and closed frames. It ranks second in ease of remembrance.

The Stewart method of analyzing continuous frames by flexure factors is neither a moment distribution, nor a slope deflection, method. For the end spans it makes use of Moment-Area Principle 1 and for interior spans it applies Moment-Area Principles 1 and 2. It involves less computation preliminary to obtaining the desired moment, but aside from that superiority it is not simpler than the Lin method.

Much is made by authors and discussers of the Grinter and Stewart methods of the point that the deflection of the structure can be visualized; but this is true of all the methods when used by competent engineers. Under dead load, live load, temperature, shrinkage, plastic flow, movement of the footings —laterally or vertically—or under earthquake movements, there is no difficulty in ascertaining the deflected shape of the structure. When, however, some, or all, of the distortion agents act simultaneously, the resultant shape is ascertainable only from the moment envelope, which may or may not be the result of the addition of moments of opposite signs. Under such load and natural phenomena combinations, the visualization advantage claimed for the Grinter and Stewart methods is not impressive and should not be a determinant in the selection of a method of analysis.

For those engineers who prefer not to tax their memories with new concepts of stiffness and the adaptation of long-known principles such as slope deflection and area moments in the alternatives to the Cross method, the latter, despite some additional calculation, will serve best for all continuous frames. For one-story frames, requiring influence lines, the modified Cross-Lin method is the fastest method of approximating moments by distribution; but review will be required before its use.

For closed frames and one-story structures, the Stewart method of traversing flexure factors will involve less computation. It will yield accurate results because it is not a method of approximation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

ENGINEERING ECONOMICS AND PUBLIC WORKS
A SYMPOSIUM

Discussion

By Messrs. Elliott J. Dent, C. Frank Allen, Bradley G. Seitz, Alfred Allen Stuart, Pierce P. Furber, R. F. Bessey, Donald M. Baker, and Phillip W. Henry

Elliott J. Dent,[61] M. Am. Soc. C. E. (by letter).[61a]—The brief discussion of flood control by Professor Mead is timely, and it is to be regretted that it was not possible to make a more extended statement. There is an urgent need that the general public be informed as to the facts as popular ideas do not include a recognition of the basic causes of flood disaster or the difficulties that must be overcome in order to effect a cure.

The occupancy of the flood-plain does not prove folly on the part of those who have selected sites subject to over-flow for their homes or their business activities. The fact is that the inherent advantages of the bottom-lands are such that they will far outweigh the disadvantage due to an over-flow that may occur once in a generation. A settler moving into a new country may have been wise in deciding that the annual gain due to the selection of a flood-plain site will enable him to set aside a reserve that will more than compensate for the loss due to infrequent flooding. A second settler may build near the first for the same sound reason, a third moves in, and eventually a town or city comes into being. As the town was expanding each individual found it to his advantage to build in the flood-plain rather than on higher ground and each individual takes the measures to guard against flood loss that he considers advisable. In spite of the object lessons afforded by the great floods of recent years, large numbers of residences and business establishments are being currently built in areas that were under water only a year or two ago.

The cost of flood protection for a single building may be far more than is warranted by the infrequent losses due to the incidence of the great floods; the

Note.—This Symposium was presented at the meeting of the Engineering-Economics and Finance Division, Boston, Mass., October 7, 1937, and published in February, 1938, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: March, 1938, by J. K. Finch, M. Am. Soc. C. E.

[61] Colonel, Corps of Engrs., U. S. Army (*Retired*), Washington, D. C.

[61a] Received by the Secretary March 1, 1938.

time, however, may arrive when the joint losses of all the occupants of a community may warrant the prosecution of a joint project for their joint protection. In some cases the losses in a number of municipalities in a single water-shed may warrant joint action with a view to the construction of reservoirs for the abatement of the floods as contrasted with the local protection works aimed at their exclusion from selected areas. When a community project can be shown to be economically justified it should be carried forward, but, pending the completion of such a project, a new comer may well find it to his advantage to establish himself in the flood-plain, reap the advantages of the site selected, and stand the loss due to an occasional over-flow. Such choice on his part may be good business, not folly.

Wherever there is a tendency for a municipality to expand in an area subject to occasional over-flow, it may be well for the Public Works Planning Agency to require that each new sub-division make provision for eventual flood protection. Such advance planning may materially reduce the ultimate cost and may thereby make it possible to execute the project at an earlier date than would otherwise be possible.

The people of the United States are accustomed to carry fire insurance and the contrast between fire and flood losses may be worth referring to in the educational campaign so badly needed at this time. The ordinary fire loss is confined to a small area; it may be limited to part of a single building; it may include the whole of a single building, or a number of adjacent buildings; a conflagration covering one or more city blocks is most exceptional. A fire loss is ordinarily very heavy for the property actually involved, and of relatively small magnitude when the wealth of the entire community is taken into consideration.

Flood losses, on the other hand, rarely involve the complete destruction of a piece of property, the average loss per premise flooded is probably less than \$500. A few weeks after the greatest flood the visible evidence may have almost completely disappeared. From their very nature, however, flood losses cover all developments within a given area which may include a considerable part of a city or of city after city along the valley of the flooded stream. A great flood may last for many days and the multitude of small losses adds up to very large figures, the numbers of persons driven out of their homes create a dramatic situation that appeals to the public. The scare head-lines about loss of life and outbreaks of disease help to raise relief funds but have little factual justification.

Fire losses are intensive and localized, flood losses are less intensive but have some of the characteristics of a widespread conflagration. Flood losses might be made the subject of reasonable insurance rates, but the insuring companies would be facing huge losses at infrequent intervals instead of the relatively well distributed losses due to the incidence of numerous small fires.

An individual planning to invest his own funds in a home or a business must decide for himself whether it is best to locate in the flood-plain, take his profits during the many years of security, and take his losses when the floods come. In the natural course of events those who elect to occupy the lowlands must accept the consequences of their choice. In due time, they may become

numerous enough and wealthy enough to make a co-operative flood-control project an economically sound public enterprise. A judicial engineering study will go far toward showing when the proper time for the execution of the project arrives. Advance planning may also result in the reservation of rights of way for use when the project is undertaken and by reducing the cost may advance the date when the work may become economically warranted.

C. Frank Allen,[62] M. Am. Soc. C. E. (by letter).[62a]—In Massachusetts, there is a law that no new railroad enterprise shall be authorized without a certificate of exigency from the proper board. There seems to be no board for that purpose in Washington, D. C., and in the projects that have been undertaken, apparently there has been no regard for what might be termed a certificate of exigency. Instead there is a certificate of emergency. Such a certificate does not reach the emergency of the project itself; it is not a reason, but an excuse, for the expenditure of huge sums of money.

Many of these projects have the element of emergency in the matter of flood control, but, in some cases, flood control has been used as a means for the encouragement of other lines of activity; for instance, power. In the creation of reservoirs for flood control, power is an antagonistic element. For power, a full reservoir is requisite; for flood control, an empty one is requisite; and it is an interesting fact that in the Dayton Flood Control System, a tablet on each of these reservoirs states that it has been designed for flood control and not for storage—in the general sense, not for power.

Engineers should be careful in accepting power accessories as coming within the exigency, or emergency, classes that are established in connection with these huge projects which, apparently, have no economic value.

Most of them have an unfortunate effect in disturbing existing agencies, and there is at present great need for stability. Nearly all of them tend to disturb existing industries, and, to that extent, in many ways, they are unfortunate. For example, why develop irrigation, when it will disturb so many of the established agricultural interests, which have already been restricted by the Government? An important feature of some of the projects is irrigation.

Bradley G. Seitz,[63] Jun. Am. Soc. C. E. (by letter).[63a]—In his paper Professor Mead maintains that the Federal public works activities of the past several years have been, in general, economically unjustified, and of little benefit to the nation. Professor Mead further contends that radical legislation has been introduced, and that the broadening of governmental activity threatens the income of investors.

The sociol-economic implications of this paper are open to question. The expansion of governmental functions is not a temporary national phenomenon to be deplored; it is the resultant of all the complex forces acting upon modern society, a force to be directed. In England, France, Russia, Sweden, Switzerland, Finland, and Norway, for example, the "Record of the Government in

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[62a] Received by the Secretary February 28, 1938.

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[63a] Received by the Secretary March 10, 1938.

Business" has been written several chapters beyond that in the United States. Most of the legislation referred to as radical has long been accepted by the conservatives of England.

The fact that some public works projects may be uneconomic does not warrant so general a condemnation. It is to be remembered, also, that conventionally sound economic analysis often leads to conclusions which are socially absurd. It cannot be applied, for example, to public education, or garbage collection. During the past few years, it has been economically unsound for industry to employ about 10 000 000 men; and yet the resulting curtailment of production in the face of a need for products, the waste of man-power and machine power, are both technically and socially appalling.

Although increased governmental activity may well result in decreased profits to the investor group, it is obvious that the condition of maximum profit to investors is frequently in sharp conflict with the general welfare. When this conflict occurs, the general welfare should prevail.

Professor Mead cites the inefficiency of the Government in business. From a social, rather than a profit, standpoint the efficiency of business itself is somewhat marred by over-expansion, duplication of effort, the apathy of mechanized workers, the production of unsound or worthless goods, the interruption of production by strikes and lock-outs, wasteful exploitation of natural resources, over-capitalization, excessive sales, advertising and distribution costs, and periodic depressions.

These defects are mentioned, not to excuse inefficiency in Government, but rather to suggest that the failure of public enterprise to satisfy the formulas of private enterprise is not, in itself, a valid criticism, since their aims are different.

ALFRED ALLEN STUART,[64] M. AM. SOC. C. E. (by letter).[64a]—The papers by Professor Riggs and Colonel Wilgus, especially, have great value, not only because of their able analyses of the subject-matter but also because they expose a great waste of public funds for projects having no appreciable value to the people at large, who must bear the cost. For these several reasons the profession owes a debt of gratitude to each of these authors for bringing to public attention such unwarranted waste of public funds. However, it must be said that the publications of the Society have such a limited circulation among all the people as to preclude the great benefit from being realized.

It is to be regretted that very many of the large works undertaken by the Federal Government are brought to the attention of Congress by incompetent individuals, and are often acted upon favorably by Congress, despite the fact that they may have no merit whatever. It is unfortunate that, as now constituted, the Society has no means of being invited by that body to advise Congress as to the merits, or demerits, of all matters presented to it which, in their very nature, requires a highly trained and experienced civil engineer to comprehend and learn the merits, and demerits, before Congress takes any action on them.

[64] Cons. Engr., Winter Park, Fla.

[64a] Received by the Secretary March 15, 1938.

In April, 1906, the writer was asked to go to Portland, Ore., to advise a banking house of New York City as to the merits of an undertaking that would require a large investment of capital. The local newspaper announced his arrival, stating that he represented large financial interests in New York. Immediately following, and continuing as long as he remained in Portland, the writer was importuned by many individuals to interest capital in numerous new developments. Many of these had much merit and could, upon completion, be made very profitable had there been a sufficient number of people to have availed of the benefits to be derived. In view of the fact that as few people were available to utilize the proposed improvements, the writer was compelled to decline even presenting them to the banking interests which he represented.

Congress is constantly having meritorious enterprises presented to it, but as now constituted that body has no means of analyzing them to discover whether they have any merit in fact.

PIERCE P. FURBER,[65] M. AM. SOC. C. E. (by letter).[65a]—One aspect of the public works question has not been stressed in the papers of this Symposium. Several references were made to the uneconomic character of Government projects, although none of the authors offered to show why this happens, or what fundamental principle is involved. The remarks were largely confined to specific criticism of individual projects as samples of Government inefficiency, and frequently to the relation between good engineering and bad economics displayed in most of them.

Often it has been said that the Government cannot operate any business successfully. Experience seems to confirm that opinion. A primary reason may be: Success in business depends upon management more than any other factor within Man's control. Finance, engineering, construction, operation—able management furnishes the impetus that drives them all. The economic success of a construction project depends primarily not on the soundness of the engineering, but upon whether the total actual cost of the completed work will bear a proper relation to the income which may be derived from its operation. This will follow if the judgment of the promoters is sound. Good management is the basis. Engineers may as well acknowledge this situation; it will do no good to cite excellent engineering design as a justification for any project. Engineering work is a means to an end, not the reason for spending money. Often, engineers themselves furnish the good management so essential to success.

Why cannot the Government match that efficient management which may be discovered within the executive branch of private enterprise? Mainly, because private business is undertaken for profit. Men who risk their capital, and their time, in a venture, naturally possess the necessary selfish interest in it to act as a spur to their wits; energy must be applied constantly to avoid loss of their own money. This applies as well to corporations, whose active stockholders exercise constant vigilance over the acts of directors and managers. Some of the direct urge may be lessened in the case of large companies with

[65] With Thomas J. McCormick, Philadelphia, Pa.

[65a] Received by the Secretary March 15, 1938.

enormous numbers of small stockholders This condition serves to emphasize the general principle as it concerns Government business, because it has often been noticed that the efficiency of large corporations which have been "orphaned" by wide dissemination of stock holdings has deteriorated in some degree due to the management no longer having a genuine stake in the company.

There is a difference between occupying an executive position of power which carries small financial responsibility, and holding a large proprietary interest in a business. The proprietor will invariably prove a better manager than his hired assistant, granting that he be not simply a financial "angel."

The chief fault in these Government projects—in any Government venture into the field of business or construction—is that no one responsible for the conduct of affairs is in any way responsible for the financial success. None of the managers, or directors, or administrators, has any stake in the enterprise. They are always spending other people's money—tax money—of which the supply always appears to be inexhaustible. The latter presumption accelerates the tendency to uneconomic operations. If only a certain sum were available, and the director knew it, there might be an incentive to try to make that sum do the work. It is fair to credit every public official with the intention to perform his task satisfactorily and efficiently. Likely enough, very few of them are consciously reckless or extravagant; they want their plans to succeed. However, they may not possess the quality of judgment required to perceive the defects. Experience of large affairs is nearly imperative for any one by whom large sums of money are to be disposed effectively. Judgments may be hampered, furthermore, by the pressure of interests (political, for instance) which should have no voice in determining policies or formulating plans for a business. Public officers are always plagued by geographical considerations which affect practically every dollar spent upon public works.

Notwithstanding the foregoing comments, the entrance of politics, local or sectional interest, or any other influences that burden the management of Government business, actually does not provide sufficient excuse for the failure of public works to pay their way. Neither is it sufficient to assert that they might be constructed or operated economically if their mangagement could be "taken out of politics." To remove all influence of that kind from the directing head would simply leave him more or less free to do as he pleased, without control from any source except public opinion or his own conscience; lack of financial responsibility would be altered in no sense. There would still be no "pocket-book" urge to caution or prudence, no risk of loss staring him in the face before every important decision.

Direct financial interest makes a difference! There may be altruistic souls who believe that a strong man may put forth the same effort on behalf of others—or the public—that he exerts in his own favor. Examples of such deserving virtue may be found in the field of pure Government or civic service, where success is not measured in money. When the cruel criterion of profit and loss enters, the situation changes remarkably. Detachment from direct financial concern cannot fail to alter any person's view of any proposition from that he would hold if his own welfare were involved. Consider trustees, for example. Honest and capable trustees for others' funds are notoriously

conservative; risks are feared, sometimes to the actual detriment of the beneficiary. Weaker men sometimes succumb to the temptation to use their positions for personal gain.

Public officials are human, as much so as trustees. They cannot possibly endow themselves with a disinterested quality, or ability to handle large sums of money in widespread business ventures which will produce results commensurate with the achievements of private persons whose self-interest and ability are all centered on one objective: To make a commercial or economic success of whatever they undertake.

Concerning particular suggestions made in the Symposium, a few comments are offered.

Advance planning is well enough to contemplate, as a theory. The prospect of arranging some program of public improvements to be undertaken in the indefinite future, when the need for work is expected to exceed the amount available, may probably now be considered good social theory. It could scarcely be considered good economic theory at any time. The only natural, or rational, economic justification for any building project is either that the construction is needed, or that the improved property will produce income. Governmental projects may be justified by the former; private, or business projects, by the latter consideration.

"Build if, and when (and only if and when) additional space is needed" is a good rule for any business. Successful and growing business applies this rule quite generally, regardless of outside conditions, much the same as large investors, such as insurance companies, handle investment funds; they buy securities when they have idle capital to invest. Waiting for a more favorable market may mean missing the market entirely, or incurring a loss of income. Postponing a building operation may mean loss of time more valuable than the possible increased cost at current prices. Advancing a building project may mean a total failure if the plant is found unnecessary when completed.

Therefore, from an economic viewpoint, success depends upon the decision of some one that the work is needed and should be undertaken at a particular time. Able management is thus the guide to proper timing, also.

Improvements included now in a program for later development during some future depression might be found inadequate, or even undesirable when the time arrives for their execution. No one can foresee the material necessities of even a few years hence. There are many examples of structures planned (at considerable expense) for future extensions, which have not been made and may never be.

The best plans are not always those upon which much time has been spent. A more important factor than time is involved in good design, or planning, namely, brains. That keen sense of discernment, which can be neither explained nor defined, which some possess and others lack, which leads some men directly to practical solutions of difficult problems, outweighs every other qualification of a prospective manager or designer. Obviously, reasonable time is necessary to any work. More time than that spent on reviewing possible methods, especially in conferences, may be the cause of inferior plans.

Advance planning, especially for long deferred projects, might easily produce just that condition.

The criticism of public works presented in this Symposium indicates not so much that dissatisfaction is expressed because the projects were hastily planned, as that most of them should never have been planned at all. All the large projects discussed are extremely questionable ventures. Perhaps none of them would have been undertaken had a serious and honest balance been drawn between economy and expediency. The least competent engineer may readily understand, without deep research, that the Florida Canal, Passamaquoddy, Grand Coulee, or TVA, will never be worth their cost if measured by the same standard ordinarily established for lesser operations. Why should not they be so judged?

Whoever attempts to justify this huge construction program as economically feasible is invited to answer or consider a hypothetical question: Could these projects ever be financed if the Government had to attract private capital instead of compelling it through taxation?

True enough, Government bonds have a market; but not on account of the security afforded by public property. They are in no sense a lien upon it. They may be regarded as a satisfactory investment for one reason only: The Government has power to tax the people to pay its debts. Failing this (if, for instance, an extravagant Government should borrow, and spend, more than it could possibly collect in taxes), only one course would be open—repudiation; because, whereas bankrupt corporations liquidate or reorganize, Governments repudiate, directly or indirectly. A Government may issue so-called money (paper notes) to retire its so-called bonds, or it may simply neglect to pay them at all, or it may refuse to pay them. Then, the bondholder has no recourse. No legal security exists for the holder of Government bonds. A bankrupt Government may not be called into Court for liquidation. The creditor's rights are determined by the debtor's pleasure, or good faith. Witness the World War loans.

This situation forces the conclusion that, ultimately, the Government itself is as irresponsible, financially, as any of its agents or officers, when considering the economic aspects of public works. It is liable only to the extent it chooses to be; it may cancel or abridge its agreements with impunity, wherever the executive has discretionary power; otherwise, by Act of Congress. As no one concerned with borrowing or spending these fabulous sums has any direct responsibility for their wise administration or expenditure, neither has the Government, as an entity, any responsibility. Executive discretion may be controlled, or legislative aims directed (after a fashion) only at the ballot box, and not there, even, in case of many appointive officers. Past official action is subject to scarcely any redress, although it may have caused serious damage.

Government extravagance differs from private waste in one important respect: The former wastes public money, the latter wastes its own. The former, therefore, is far more reprehensible, contrary to the cynical notion many people entertain that the Treasury is a huge grab-bag, a bottomless mine to be worked by clever people for their own benefit.

Any unnecessary public works are a drain on the national wealth and welfare. Business projects conceived in false economy, and executed extravagantly, will lead a Government as surely into bankruptcy as similar mismanagement will wreck a private business. The principle is the same, the distinction being one of degree. The evil day may be longer deferred by a Government's using powers denied to private persons.

Whether the Government programs will constitute a national policy, or portend increasing public ownership and operation of business probably depends on how long the people may take to realize the extent of the uneconomic works already under way, and that they are to pay the cost themselves. When they have reached the inevitable conclusion that the management of public business is irresponsible, under all circumstances, they should understand the futility of entrusting economic problems to Government. The necessary functions of pure government (national defense, protection of public safety, etc.) fall into an entirely different category.

The Engineering Profession may serve the country by hastening to develop the national understanding of this momentous question. Its members should view the subject patriotically, and express their deliberate opinions with a single purpose for the best interest of the entire country. It is a subject fit to challenge one's devotion to the truth; a test for one's claim to high ideals. Let engineers hold to the course of reason and common sense, no matter what ignorance or sophistry may argue to the contrary.

Colonel Wilgus cites the vision of certain builders, cautioning that vision is an attribute of unknown value. It is one element of the sense, or judgment, emphasized herein. Vision, however, varies with the point of view. Another hypothetical question is submitted: Would any sensible, level-headed manager, or board of directors—any that ever lived—use their own funds to build these public works if they, themselves, had to assume the risk of possible failure with the attendant losses? (Assume, of course, that they would be entitled to whatever profits might accrue.)

Is it not a question of whose vision to trust? Unless vision is accompanied by substantial financial capacity it seldom inspires confidence. The Canadian Pacific Railroad and the Grand Central Terminal illustrate the point admirably. In these projects, vision, backed by capital, carried responsibility. The vision that promulgated the Florida Canal was devoid of it.

Apparently, some projects have been ordered built in spite of adverse reports by competent engineers. Such proceedings confirm the belief that lack of financial responsibility leads to recklessness, inefficiency, and waste. A responsible executive would hesitate before committing himself or his company to a policy involving large expenditures after receiving not only one, but perhaps several, very discouraging expert reports upon the economic feasibility of a proposition.

The inception of these public works brings to mind the figure of Peer Gynt, in Act IV, Scene V, of Henrik Ibsen's play. Stranded in Morocco, Peter Gynt has wandered inland to a place overlooking the Sahara Desert. After a soliloquy as to the "enormous boundless waste" stretching out before him, he is inspired by an idea:

"A dam! Then I might ——! The hills are low.
A dam! Then a cutting—a canal—
And through the gap the rushing waters
Would fill the desert with a life-flood."

Then, his thoughts rushing out with a frenzy of idealistic zeal, he visualizes town after town growing up in the Sahara Desert—new industries, new trade, new discoveries; and, finally, with settled conviction, he springs up:

"I only need
Some capital, and the thing is done—
A golden key, and the ocean's gate
Is open! * * *"

R. F. Bessey,[66] M. Am. Soc. C. E. (by letter).[66a]—This Symposium is disappointing because, essentially, the papers (except that by Mr. Fay) represent only one viewpoint of the subject. With Mr. Fay's suggestions for orderly investigation and planning of public works the profession should be in hearty accord. He also suggests, in developing his thesis, some of the fundamental national economic considerations relating to public works, namely, unemployment, the business cycle, national income, the construction industry, public finance, etc.

As to the remainder of the Symposium, the dominant note is one of criticism and the general inference is that current programs for large public works are uneconomic and unsound. With the constructive suggestions, that public works economics should be studied and discussed by the Engineering Profession, there should be general agreement.

It is not the purpose of the writer to undertake a detailed defense of the projects questioned. Rather, it is his intention to urge a deeper and more general consideration of the economics of public works by all engineers, and briefly to indicate some of the broader economic considerations relating to the public works of the Pacific Northwest.

It is hoped that the Symposium will bring more professional thought to the fundamental economic and social factors—in addition to the purely physical and financial factors—in the design of public works. If engineers are to be of optimum value to society, and are to take the valuable place in public affairs that their basic training warrants, they should acquire a certain knowledge and appreciation of economic, social, and governmental factors. It should be understood, however, that public works is not the field of the engineer exclusively; the engineer must learn to work co-operatively with the economists, the social and political scientists, and the legislators and administrators, who are also vitally concerned.

It is obvious that the economic factors relating to public works are somewhat different from those of private engineering works; also that the factors for the large public projects may be somewhat different from those of the smaller ones. Essentially, the test of the economic feasibility of the private work is profit or loss. Essentially, the test of public work is the extent of public benefit. This

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[66a] Received by the Secretary March 19, 1938.

is public profit or loss, but all the items are not necessarily measurable in dollars and cents. The smaller public works are quite often designed for a single purpose and the tests of economic merit may be relatively simple: Does the project provide the particular improvement or service needed? Does it fit into a comprehensive plan? Is it physically feasible? Is it legally and administratively feasible? Can it be financed? Will the value of service cover the cost, including capital, interest and amortization, maintenance and operating expenses?

The larger projects involve these tests also; but, in addition, other factors (many of them of a more imponderable nature) must also be taken into consideration. For example, the most important test of the need of a water supply system may be whether a great metropolitan area can continue to grow, or even survive, without such a system. With respect to a slum clearance and housing development, the tests are not solely whether rentals will pay the fixed and operating charges, but whether a community can afford to continue to pay the estimably greater upkeep cost of the blighted area or the inestimably great social costs of the slums.

Public works should be considered as one of the available means of economic adjustment—not only with respect to unemployment of labor, industry, and capital, and the swings of the business cycle, but in correction of, or compensation for, community or regional deficiencies, or unfavorable economic conditions or balances.

Without attempting to elaborate, criteria for larger public works would involve, in addition to comparison of cost and income, or cost and direct benefits, the weighing of various socio-economic factors, including their effect on: (*a*) The stability or movement of population; (*b*) the flow of wealth; (*c*) the economic activities of the community; (*d*) patterns and standards of life; (*e*) public welfare—health, education, safety, recreation, etc.; (*f*) other programs and projects; and (*g*) the conservation and beneficial use of resources, including those of land, soil, water, minerals, etc. Some of these effects might be illustrated by brief discussion of one or two classes of public programs and public projects.

Irrigation development permits a more intensive and stable use of land. In some areas of the United States, very large and important segments of rural and urban civilization would wane and many communities would disappear without it. Irrigation development also involves such economic considerations as movements of population, changes in the pattern of agricultural production, new agricultural industries, and new rural markets. Therefore, the tests of the economic merit for an irrigation project are not merely whether the water users pay their fees and amortize the cost on time; the more indirect effects, the community wealth created, the taxes paid, etc., must also be considered. Reference to a study of the Washington State Planning Council on this subject[67] should broaden the view of some of those who may look at irrigation economics only in terms of project cost and direct repayment. The report of the Council points out that the Yakima Valley, developed through irrigation, produces as much as $50 000 000 annually in farm products, livestock, and value added to

[67] "Reclamation—A Sound National Policy, as Demonstrated by the Yakima Valley and Other Irrigated Areas in Washington," Washington State Planning Council, 1936.

raw materials through manufacture; also that, of the new wealth, produced annually, more than one-half is expended for the products of Eastern industries.

Projects for water supply, water conservation, etc., have high importance in any region, but in the Western United States civilization cannot advance, or even be maintained at present levels, without them. Conditions in the high plains[68] point to the great importance of such projects, and also emphasize the principle that in the planning, design, and financing of public works, the fundamental social and economic needs of the community should be determined and such resources as are available for public works should be devoted to meeting these needs in proportion to their importance and urgency, rather than on the basis of custom or precedent.

The economics of hydro-electric power development involve much more than a comparison of costs of fuel-generated and water power; such factors as the conservation of a national fuel and chemical supply (the dwindling oil reserves, for example), and the auxiliary effects of river control (in navigation or irrigation, for example), must also be weighed.

Pointing this discussion more specifically toward the economics of large Pacific Northwest projects concerning which questions are raised in the Symposium: The large projects in the Pacific Northwest are essential to this relatively young region, which cannot meet some of the present needs and cannot advance materially without measures of this kind.

Land is a fundamental factor in the Pacific Northwest, as elsewhere. The Pacific Northwest[69] includes 300 000 to 350 000 sq miles, depending on how it is defined. The four Pacific Northwest States of Washington, Oregon, Idaho, and Montana, contain nearly 400 000 sq miles, about 13% of the area of the United States, and less than 3% of the national population. The crop land in these States is limited in extent; it is only about 26 000 000 acres out of a total of 254 000 000, and its productive capacity is generally limited without water.

Lands at present under irrigation total about 5 000 000 acres. There are feasibly irrigable as many as 6 000 000 acres more. Of this area, about 1 200 000 acres lie in the Columbia River Basin, under the Grand Coulee Project. The net additional total of crop land feasibly reclaimable by other means—drainage, diking, stump clearing—is not definitely known, but perhaps would total an additional 2 000 000 to 4 000 000 acres.

Reclamation in the Pacific Northwest also bears an important relationship to national land needs and programs.[70] For example, it should be considered in relation to the indicated need of retirement, nationally, of about 25 000 000 acres from cropping as a soil conservation measure and to 30 000 000 acres, or more, which will be required during the next generation to meet the needs of new population. In addition, or perhaps tending to offset technological advances in land use, there undoubtedly will be a large land requirement during the next generation to meet the growing need of production of industrial raw

[68] "The Future of the Great Plains," Rept. of the Great Plains Committee, December, 1936.

[69] "Regional Planning," Pt. 1, "The Pacific Northwest" (Columbia Basin Rept.), National Resources Committee, 1936.

[70] Rept. of National Resources Board, December, 1934; also, Supplementary Rept. of the Land Planning Committee.

materials. Mr. J. W. Haw discussed some of the national economic factors in land reclamation before the Annual Convention of the Society in 1936.[71]

In the Pacific West there is a land problem of particular urgency. In 1936 alone, conservative checks and estimates indicate that 10 000 additional families sought settlement on the land in Washington, Oregon, Idaho, and Western Montana. Similar checks and estimates for the first half of 1937 indicate that the totals for that year will be even larger than in 1936. Various factors, such as climatic and economic conditions and age composition of populations elsewhere as well as past trends, indicate that such westward migrations are likely to continue. Of course, all these rural migrants will not settle on irrigated land, although the majority undoubtedly would do so if opportunity offered; but in two years, allowing only 40 acres per family, there were enough migrants to settle the equivalent of about one-seventh of all the feasibly irrigable land in the region to which they have come. In other words, there was a sufficient number of families in two years to settle the equivalent of at least two-thirds of the land of the Grand Coulee reclamation project, which may require twenty years for completion. It might be added that, lacking a comprehensive program of land reclamation, less than 20% of the immigrant farm families in the Pacific Northwest in 1936 were able to settle permanently on economic farm units.

Additional factors affecting the economics of Western public works lie in the national ownership of a large part of the land resources. (More than one-half the land area of the eleven Western States is in Federal ownership.)

Hydro-electric power is one of the outstanding resources of the Pacific Northwest, which contains about 40% of the potential hydro-electric power in the United States. The wise and the earliest practicable economic utilization of this non-exhausting natural resource is naturally of great importance to the people of this region and to the nation. In round figures, the potential hydro-electric power capacity of the Pacific Northwest is about 16 000 000 kw. The developed power capacity is about 1 800 000 kw for four States—Washington, Oregon, Idaho, and Montana. The initial capacity at Bonneville (two units) will add 86 400 kw, and the ultimate added at Bonneville will be approximately 500 000 kw. The initial capacity at Grand Coulee will probably be 315 000 kw, and the ultimate planned is 1 890 000 kw. In round figures, the production (four States), in kilowatt-hours, increased from a negligible amount in 1909 to 900 000 000 in 1910, to 3 000 000 000 in 1920; and to 6 000 000 000 in 1929. Following a drop below 5 000 000 000 in 1932, production again advanced to nearly 7 500 000 000 in 1937. The probable production, on the completion of Bonneville, Grand Coulee, and other plants, now under way or planned (1938), will be about 18 000 000 000 kw-hr. Curves of power consumption may be projected into the future on one's own optimistic, pessimistic, or average basis; neither future compounding rate of increase, rate of diminution of the compounding rate, nor the ceiling level can be estimated with any certainty. So many factors are involved that projection of any one curve for more than two decades or so is scarcely warranted; it would be more realistic to try and

[71] "Irrigation and the Land-Use Program," by John W. Haw, *Civil Engineering*, October, 1936, p. 663.

estimate curves marking upper and lower limits for such long-range use. In any case, however, it should be apparent that there are still many potentialities for expanded and new uses of electric energy, that saturation is not yet in sight, and that if the country advances at all—that is, barring economic ineptitude on the part of the nation and its people—the power will be used, and probably during a time not inconsistent with the probable rate of completion of the current Columbia River power projects.

With reference to Bonneville, the Oregon State Planning Board has conservatively estimated that the assumed Oregon half of Bonneville's ultimate capacity will be absorbed by its Oregon market area, and that 200 000 kw more will be required in sections of the State outside the Columbia Basin area by 1946.[72]

The power projects in the Pacific Northwest should be considered also as potential keys to a needed broader industrial development and a better-balanced regional economy. Complete self-sufficiency is not thought of as a desirable or practical ideal, but studies indicate, for the Pacific Northwest, a marked deficiency in manufacturing activity and a tendency to "live off" of capital through primary economic dependence upon extraction of regional resources which are not readily replaceable—primarily those of soil, forests, and minerals. The fundamental condition, which indicates a flow of wealth to the industrial and financial centers of the East, may be seen at a glance in Fig. 1 which shows the net values of various classes of imports to, and exports from, the region. For a secure region and a secure nation there must be an adequate return flow of wealth. Public works is one of the means of bringing about such a flow. If these public works tend to provide for improvement of the economy of the region, as well as for an immediate return of wealth, so much the better. Abundant and low-cost power, tending to increase industrial activity, payrolls, and dividends, should be highly important to the Northwest economy.

Incidentally, another form of return flow of wealth is tourist travel, which, it will be noted, is also somewhat related to public works through highway construction, national park, and national forest programs.

The Pacific Northwest is one of the regions in which inland navigation is of high present and potential importance. The regional freight rates are among the highest in the United States. Further economic transportation development will be essential to the general development of the region. Modern trunk inland navigation is an important corollary of the land, power, and industrial developments. Both East-and-West and North-and-South trunk lines are feasible from the engineering standpoint. Providing efficient transportation for bulk materials in particular, inland waterway improvements, as found economically feasible, will aid in the further utilization of regional resources. The juxtaposition of economical power and ocean navigation in the Lower and Middle Columbia River is particularly significant. Furthermore, navigation is one of the multiple uses of the water resources that, together, make large-scale development economically feasible.

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[72] "Use of Electricity in Oregon with Forecasts of Future Demands," Oregon State Planning Board, September, 1936.

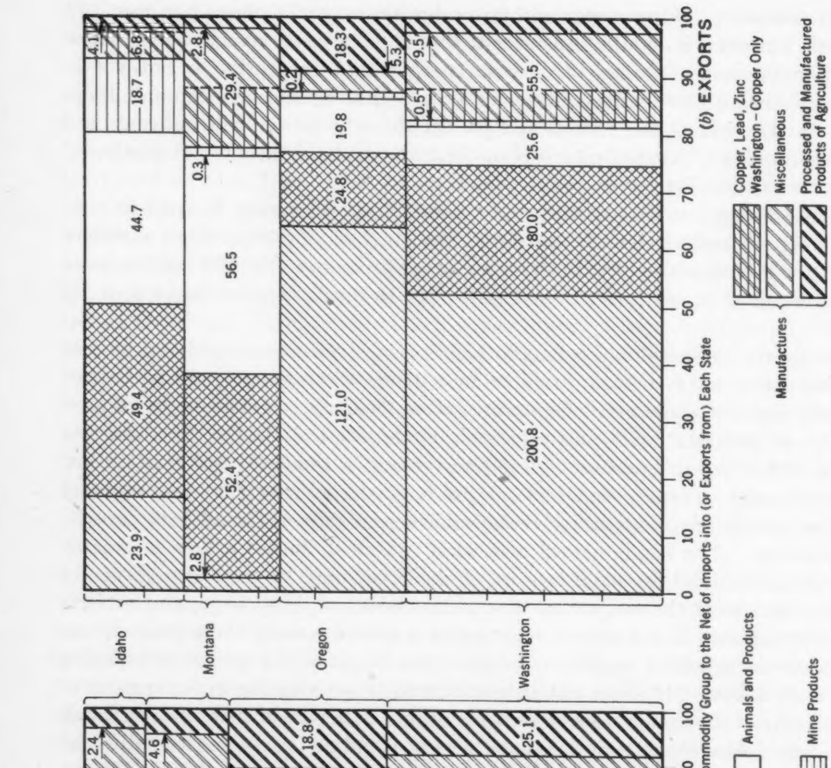

FIG. 1.—ESTIMATED VALUE OF APPARENT NET IMPORTS AND EXPORTS OF THOSE COMMODITIES SHOWING AN IMPORT (AND EXPORT) BALANCE IN THE EXTERNAL TRADES OF THE PACIFIC NORTHWEST STATES, 1929

In connection with multiple water use it should be noted that the Grand Coulee Project, with its more than 5 000 000 acre-ft of useful storage, will provide, in addition to reclamation and power, general benefits in river regulation—primarily, in increasing ultimate efficiency of all down-stream power plants and, secondarily, in flood control and river navigation.

There are other large projects[73] in the Pacific Northwest in various stages of development; for example: Various water conservation and irrigation projects in Washington, Oregon, Idaho, and Montana; further projects in the comprehensive plan for the Lower and Middle Columbia and Snake Rivers; and the Willamette Valley project in Oregon. (Comprehensive studies and reports on this last-mentioned multiple-use project have been made by the Corps of Engineers, U. S. Army, and by the Oregon State Planning Board.)

It is strongly believed that there must be a place in any long-term public works program for the large projects of this character. It is also believed that matters of law and appropriation, rather than demonstration of sound economics, will in most cases be the last hurdle to be topped in getting them under way. Properly conceived, such projects have far greater and more lasting economic effects than small, unrelated public works projects. Ranking in economic importance with the large projects are well co-ordinated programs of smaller public works (for example, such programs as that of the Montana Water Conservation Board, being carried on in co-operation with Federal Public Works and Works Progress Administrations). Such projects and programs are important for their regenerative as well as direct effects—through the foundations provided for an improved or more stabilized community economy or through the stimulus provided for other economic activities.

The programs of the Pacific Northwest have been effective in the employment of labor and of the construction industry. The larger projects have also been particularly effective in providing employment of men and industry in the East in the manufacture and shipment of equipment and supplies. It is believed that few engineers of the Pacific Northwest would doubt the vital importance of these projects to the members of the Engineering Profession in the region.

In connection with the economics of unemployment relief through public works, there must be considered such factors as beneficial effects in industrial activity, national income, national wealth, etc. It should also be borne in mind, that "write-off" to unemployment, where there is any, is generally less in the larger projects and programs than in the smaller works programs and projects, in spite of the fact that these large projects are also effective in emergency relief. Certainly, the savings which they have permitted in direct and indirect relief should be included among the economic factors relating to the large projects.

In conclusion, the writer would like to add a little to the challenge of the Symposium on public works economics. It is hoped that the Engineering Profession at large will be challenged to give a greater consideration to the economic and social bases and consequences of its work. It is hoped that the

[73] "Drainage Basin Problems and Programs," National Resources Committee, 1936–1937.

present and future leaders and educators of the profession will have a constantly broadening concept of engineering economics in relation to public works, and of the responsibility of the profession in this field. It is believed that Mr. Fay's constructive paper presents some of the logical avenues to such consideration by the profession.

DONALD M. BAKER,[74] M. AM. SOC. C. E. (by letter).[74a]—Two lessons may be drawn from the four papers comprising this Symposium—the importance of the Construction Industry in the national economy, and the necessity of advance planning and programming of public works projects. When an industry that accounts for an annual expenditure equal to 15% of the annual income (as did the Construction Industry during the five pre-depression years, 1925–1929) evidences such an abrupt decline as occurred after 1930, the resulting effect upon economic conditions is certain to be felt keenly.

Data as to expenditures for construction work presented by Mr. Fay have been re-arranged in Table 5 for the five pre-depression years, 1925–1929,

TABLE 5.—ANALYSIS OF CONSTRUCTION EXPENDITURES, 1925–1934

Agency	Average Annual Expenditures, 1925–1929			Average Annual Expenditures, 1932–1935			Per Capita Expenditures, in Dollars		Ratio, Column (4) / Column (1)
	Millions of dollars	Percentage of totals	Percentage of public works	Millions of dollars	Percentage of totals	Percentage of public works	1925–1929	1932–1935	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Public Works:									
Cities	1 364	11.5	48.2	563	13.3	29.0	11.54	4.47	0.413
Counties	746	6.3	26.4	125	3.0	6.4	6.31	0.99	0.168
State	466	3.9	16.5	450	10.7	23.2	3.94	3.57	0.966
Federal	252	2.1	8.9	802	19.0	41.4	2.13	6.36	3.185
Totals:									
Public works	2 828	23.8	100.0	1 940	46.0	100.0	23.92	15.39	0.686
Private	5 242	44.0	185.1	825	19.5	42.6	44.30	6.55	0.158
Public utilities	3 830	32.2	135.3	1 455	34.5	75.0	32.40	11.55	0.380
Grand total	11 900	100.0	420.0	4 220	100.0	217.5	100.62	33.49	0.255
Total, private plus public utilities	9 072	76.2	320.8	2 280	54.0	117.5	76.70	18.10	0.251
Average, annual national income	78 343	....	....	46 472	....	....	....	....	0.592
Average annual per capita income	663	....	....	368	....	....	....	....	0.552

inclusive, and the four depression years, 1932–1935, inclusive, in order to afford a comparison between pre-depression and depression construction activities. This analysis emphasizes the fact that private and public utility construction was the "backbone" of the industry prior to the depression, accounting for more than three-fourths of the average annual expenditures during that period. In spite of largely increased public expenditures during the four depression years, public utilities accounted for one-third of the expenditures, and private construction for one-fifth.

[74] Cons. Engr., Los Angeles, Calif.

[74a] Received by the Secretary March 28, 1938.

The greatest proportionate reduction in expenditures occurred in those made by private agencies and by counties, where they fell to about one-sixth of pre-depression rates. Activities of cities and public utilities were reduced three-fifths, those of States showed a slight decrease, and those of the Federal Government increased threefold. In spite of the increased Federal expenditures, public works decreased one-third, and total construction activities were reduced to about one-third of the pre-depression rate.

Data in Table 6, taken from 257 identical cities, indicate that the great reduction in private construction was probably due in large measure to the almost complete cessation of residential building, which was caused by widespread unemployment, reduced incomes, and uncertainty as to the future of those still employed. Counties usually carry on their construction programs from current revenues received from taxation. Deflation of assessed values during the early years of the depression no doubt had a telling effect upon revenues, and the necessity of caring for the local relief load as well as maintaining other functions of county government left little surplus for public works projects. A considerable proportion of the construction expenditures of both cities and public utilities usually goes for expansion of facilities to care for increase in population. The "back to the land" movement, which was very active during the early years of the depression, slowed down urban population increase to a great degree, and both city and public utility construction was probably gaged closely to that necessary for operation and maintenance.

TABLE 6.—Building Construction in 257 Identical Cities

Description	1925–1929	1932–1935	Ratio, $\frac{1932\text{–}1935}{1925\text{–}1929}$
New residential construction per year	\$1 983 217 000	\$120 777 000	0.0608
Families provided for per year	398 521	32 784	0.0822
Expenditures per family	\$4 975	\$3 692	0.7420
Total building construction, including additions, alterations, repairs, and other buildings per year	\$3 516 282 000	\$473 922 000	0.1347

A very large proportion of State expenditures is for highways, financed by gasoline and motor vehicle taxes, and bond issues. Since people still operated their automobiles during the depression, and a large State bond issue takes considerable time to spend, it could be expected that State expenditures for construction would be maintained. In the light of what occurred there remained no agency other than the Federal Government financially able to make an attempt to support the Construction Industry. It was as unprepared to embark upon a large program of public works as it was to enter the World War in 1917; but so was nearly every other agency engaged in construction, except possibly the various States, which, in general, have developed the practice of advance planning and programming of highways, because of the certainty of future income. City planning has become a recognized function of many city governments since 1920, but most city planning boards or commissions have either become involved in the complexities of zoning, or if they have developed

anything approaching a comprehensive city plan, have given little attention or thought to the financial or timing aspects of achieving it.

Considerable criticism is offered in the papers of the Symposium to the economic aspects of certain large public works projects embraced in the Federal program. Although not attempting to defend any of the projects referred to, the writer would like to call attention to several facts which must be borne in mind when they are being discussed. These facts are:

(*a*) Lack of preparedness and of planning always causes waste, in public works programs, war, or other activities.

(*b*) Public works projects are seldom initiated or promoted by politicians or public officials. They are usually conceived, and in their early stages promoted, by local interests that expect, individually, to benefit by their construction or operation. This, in itself, is not at all reprehensible in the case of sound projects. Were this not the practice, there would be a far less number of worth while public works projects. These local interests include engineers, architects, material and supply firms, contractors, land-owners, industrialists, financiers, etc., acting individually or through Chambers of Commerce, Boards of Trade, and professional or trade associations. The project is then "sold" to the local public, and becomes a local issue. Once it attains this status, politicians and public officials, under the influence of pressure groups, adopt and support it. Let them question the need or economic feasibility of the project, and other aspirants for public office will be found in large numbers ready to come forward with offers of support if given an opportunity to occupy public office. The writer agrees with the opening statement of Professor Mead that "a well informed public can and will raise such objection to projects which they know to be unsound that no Federal, State, or Municipal Administration will dare to undertake such projects," but wonders how the public is to be informed. He knows of instances where individual engineers and engineering organizations—even Sections of the Society—have expressed disapproval of the economic feasibility of public works projects in serious and carefully prepared reports, only to have their voices drowned out by the loud acclaim of the local public (not the politicians and public officials), this acclaim, in many cases, being based upon other favorable reports made by engineers. There may be some satisfaction in keeping the record clean, even if it is buried, but until engineers learn to adopt the technique of the promoters in gaining a public audience for their conclusions upon uneconomic public works projects, their influence in heading off such projects will be little felt.

(*c*) The amount of attention paid to the economic feasibility or need of any public works project by the general public of a community benefited by its construction or operation is usually in the inverse ratio to the proportionate cost of the project, or of its operation paid by the community. Projects financed by local taxation or bond issues primarily are likely to be widely discussed, pro and con, and criticisms of unwise projects usually find an audience. However, where costs are borne indirectly to a large extent, or are met from a distant source, the customary tendency is not to "look a gift horse in the mouth." This points to the advisability of local participation to the greatest degree possible in the costs of public works projects.

Fig. 2 shows the cumulative expenditures for construction for the 11-yr period, 1925 to 1935, inclusive, according to various classifications as given by Mr. Fay in Tables 1 to 3, and indicates a deficiency in expenditures by 1935 of $36 760 000 000 based upon the pre-depression rate of $11 900 000 000 per yr; or $100.62 per capita per yr. This deficit is classified as follows:

Public works	$ 3 150 000 000
Private	23 480 000 000
Public utilities	10 130 000 000
Total	$36 760 000 000

and would indicate annual expenditures of the order of $16 000 000 000 for the 10 yr following 1935 if the deficit is to be overcome and the pre-depression rate resumed. The writer is inclined to believe, however, that the pre-de-

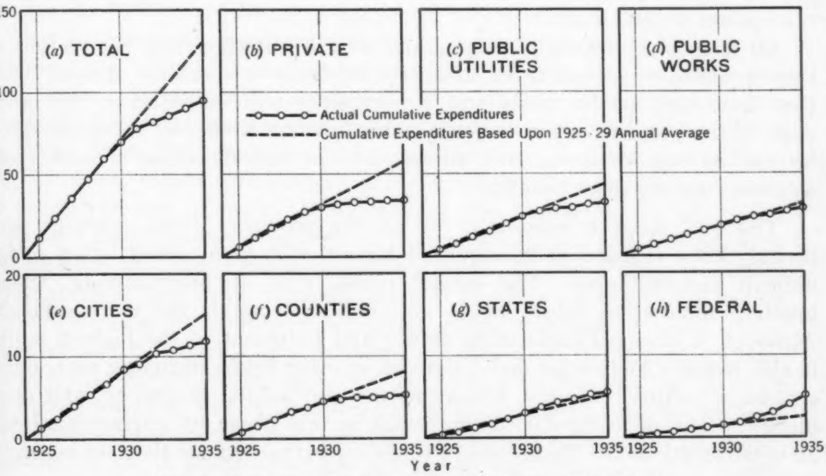

FIG. 2.—EXPENDITURES FOR CONSTRUCTION, 1925–1935

pression rate of $100.62 per capita was artificially stimulated by rapid increase in urban population and an urgent need to supply the population of the nation with a greatly increased standard of living. On the basis of a national population increase of 7 224 000 during that 5-yr period, it meant an expenditure of $8 240 for each person added to the population. However future construction expenditures are computed, they will be large, and the great probability is that those allocated to public works will embrace a continually increasing proportion. This trend was well established prior to the depression and has been greatly fostered during depression years, and such trends are persistent.

Since all construction work is undertaken to serve one or more of the following purposes: (*a*) To replace existing facilities which have worn out or become obsolete; (*b*) to provide new facilities to care for increases in population; and (*c*) to provide new facilities necessary to increase the standards and conveniences of living, it is probable that an analysis of construction expendi-

tures made during the five pre-depression years would show that many of them were made for Purposes (*b*) and (*c*). Indications are to the effect that the trend toward further urbanization, so strong during the 1920–1930 decade, may slacken in the future. This trend, with an approach to stationary national population and a probable reduction of westward migration, would indicate that future construction will be made primarily to serve Purposes (*a*) and (*c*).

If, as it would appear, a large program of public works construction is indicated in the future, the matter of planning and programming in connection with such program takes on added importance. Such activities must be conducted not only on a national basis, but by State and local agencies. Strictly speaking, they fall into two phases:

(1) The development of physical plans for public works projects, which will result in an effective and efficient utilization and control of natural resources for the public benefit, or will serve the convenience and necessity of the general public; and,

(2) The development of a program for the initiation and completion of these projects which will bring them into existence or operation at such times that increasing public needs and requirements will be satisfied, will keep costs of the projects within the reasonable financial ability of those benefited by and paying for them, and will provide for the allocation of such costs equitably among those benefited.

The first phase is essentially within the province of the engineer, and he has, when engaged in it, acquitted himself with great credit, often under difficult circumstances. The second phase, that of programming, which involves forecasting future needs and requirements, is far more difficult. Although it involves engineering ability and judgment of the highest order, it also requires knowledge and judgment in other fields, including economics, sociology, politics, industry, human nature and habits, geography, and even history. The development of leadership in this phase by engineers affords an opportunity to the Engineering Profession second to none that has occurred heretofore.

In illustration of what is meant by the foregoing two phases of activity in public works planning, consider the case of a large drainage basin which offers possibilities of development for hydro-electric power, irrigation, domestic water supply, and recreation. It is one thing for an engineer to prepare plans and make designs for the various dams, conduits, power-houses, transmission lines, etc., and to build them; but it is another thing for him to plan these structures of proper sizes and to recommend the date for their construction at such times that, when they are placed in operation, fixed charges for unused capacity will not be burdensome, that demands for their services can be met, and that costs will be allocated equitably among those benefited or served. The writer has noticed a tendency on the part of too many engineers to devote 90% of the contents of their reports to the subject of physical plans, and only 10% to the program of development, allocation of costs, etc.

Because of changes that occurred during the 1920–1930 decade, and which will occur because of the depression, it will be more difficult to make predictions

of future needs and requirements than in the past. Indications are that many trend curves developed a point of discontinuity about the year 1930.

The problem of national population is an excellent example. Forecasts of probable national population increase[75] indicate a numerical growth between 1930 and 1960 (30 years) but slightly in excess of that which occurred during the 10-yr period, 1920 to 1930. In 1960, it is estimated that 42% of the population will be more than 40 yr of age, as against only 28% in 1930. The effect of this upon habits, needs, and methods of living must be considered. During the 1920–1930 decade the population of urban places increased 27%, whereas it is estimated that during the three decades, 1930–1960, even maintaining the rate of migration to urban areas that occurred during 1920–1930, the total increase in population will be only 19.5%, or 6.5% per decade. If these forecasts are correct, the effect upon such construction activities as residential building, public utilities, etc., will be pronounced. Migration of people has always been a far more important factor in population increase in the Western States than natural increase. This migration follows a law similar to that of Ohm's law in electricity—the rate of migration varies directly with differences in economic pressure at its origin and its destination, and inversely with the resistance to the movement of people. People move away from places where opportunities are few and where it is difficult to make a living, to places where opportunities are greater and living easier, provided they can get away. During the depression the "difference in potential" between the East and the West has been greatly reduced, and the difficulty of getting away and starting to move has increased. Whether these conditions are permanent or temporary is something that is still not clear, but it should be considered seriously by those making long-range programs of public works.

Another factor that must be appreciated and considered in making programs is the fact that curves of per capita production and consumption of many of those things which go to the improvement of standards and conveniences of living are showing tendencies to flatten out, indicating a condition of approaching saturation and a probable future production and consumption, at a rate of increase more closely paralleling that of increase in population. The data in Table 7 are illustrative of this point.

TABLE 7.—Relation Between Consumption and Population Increase

Description	1920	1925	1930	1935
Electric power consumption, in kilowatt-hour per capita	408	573	778	....
Motor vehicles per 1 000 population	87	174	215	....
Telephones (Bell System) per 1 000 population	111	139	163	....
Radios per 1 000 population	....	....	109	169

These factors have had a most important effect upon the thinking and living habits of the American people, and were no doubt responsible to a considerable extent for the pre-depression construction activities.

There is no reason why the engineer should not assume leadership not only in planning but in programming public works projects; in fact, there is

[75] Rept. of National Resources Committee, 1934–1935, pp. 95–97.

every reason why he should, but he must realize, when he attempts to do so, that technical engineering is only a part of his work, and if he enters this phase, he must be ready to assume the blame when the projects for which he is responsible develop into economic failures, when judged by proper standards, just as he now assumes the blame for physical failures.

PHILIP W. HENRY,[76] M. AM. SOC. C. E. (by letter).[76a]—Among the "notable examples of admirably run publicly-owned" revenue-producing enterprises, Colonel Wilgus cites the municipal subways owned and operated by the City of New York, N. Y., but he does not mention that they are not earning interest charges. The original rapid transit systems (elevated railways and subways) were built by private capital on which they made a reasonable return; but no capitalist would have under-written the municipal subways which have not only failed to earn interest, but, through competition, have greatly decreased the earnings of privately-owned subways. Had the City of New York left the transit field to private capital, it would not now be levying a 2% sales tax and looking around for other methods of taxation in order to balance its budget. In building subways the City Government was no more interested in the economic aspects of the case than was the Federal Government when it authorized the construction of Bonneville, Fort Peck, Grand Coulee, and other projects.

The capitalist asks of a given project: "Will it make a return in dividends?" The politician asks: "Will it make a return in votes?" Under the capitalistic system Americans have attained the highest standard of living known to history. Under the political system, if adopted generally, they will see this standard stationary or even lowered.

[76] Cons. Engr., New York, N. Y.

[76a] Received by the Secretary March 31, 1938.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLOOD ROUTING

Discussion

By Messrs. Cecil S. Camp, and R. D. Goodrich

Cecil S. Camp,[10] Jun. Am. Soc. C. E. (by letter).[10a]—The method of routing floods through reservoirs, presented in this paper, should be of value to all engineers concerned with analyzing the effect of various dams, or groups of dams, on the control of floods. The method can also be used to advantage in determining the operating schedule of a system of flood-control or navigation dams. Because of the many data on stream cross-sections and profiles required for its use, the method will probably be of more value in studies of large river systems than for single dams.

It was rather surprising to the writer that the routed natural hydrographs agreed so well with the actual stream-flow records when it is considered that there was considerable variation in the extent of the area for which local and tributary inflow was calculated from rainfall for the various floods studied. This variation was due to some stream-gaging stations not being in operation for the earlier floods studied. However, the fact that there was considerable variation in this area serves to emphasize the adaptability and usefulness of the method.

The necessity of obtaining accurate stage-discharge relations, covering the maximum possible range of these values, at the different "routing stations" cannot be emphasized too strongly. The effect of channel storage between the "routing station" and the stream-gaging stations which are used in the construction of the rating curve, must be given careful consideration. Furthermore, in those cases where the submergence factor between a down-stream station and the "routing station" is used, the rating curve must be constructed to cover the extreme values of the submergence factor that may occur when the stage at the down-stream station is affected by excessive run-off from a tributary below the "routing station," or by confluence with another stream. In studies of the 1937 flood on the Lower Tennessee River the writer found that the

Note.—The paper by Edward J. Rutter, Assoc. M. Am. Soc. C. E., and Quintin B. Graves, and Franklin F. Snyder, Juniors, Am. Soc. C. E., was published in February, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order that the opinions expressed may be brought before all members for further discussion of the paper.

[10] Asst. Prof. of Civ. Eng., Coll. of Applied Science, Syracuse Univ., Syracuse, N. Y.

[10a] Received by the Secretary March 8, 1938.

effect of the extremely high stages on the Ohio River, at Paducah, Ky., was such that it was impossible to check observed stages at the Gilbertsville, Ky., dam site by the use of a Gilbertsville rating curve, which had given very good results when used for several other floods in which the Paducah stages had not been so high.

R. D. GOODRICH[11] M. AM. SOC. C. E. (by letter).[11a]—The importance of flood routing is becoming greater with every added disaster and a year seldom passes without serious loss and damage in some quarter from this cause. Hence, an accurate and detailed description and record of methods, as given by Messrs. Rutter, Graves, and Snyder, is a valuable contribution to the literature on floods.

The reliability of discharge estimates and flood frequencies based on such computations as those described by the authors may be questioned by some and, therefore, when predictions based on similar studies are confirmed by later experience, one's confidence in these methods seems to be justified.

A case in point occurred with the floods of December, 1937, on the Sacramento River, in California. It was while engaged on a detailed study of the Sacramento Valley Flood Control Project and the probable increased protection that would be provided by the construction of the Kennet Dam (Shasta Reservoir) that the writer developed the solution of the flow-storage relation used on the Tennessee River investigation. The flood frequencies computed from the results of routing floods on the Sacramento River indicated that the location where flood damage was most probable was at Butte City, Calif., with an average probability that the capacity of the project would be equalled or exceeded about once in twenty-one years.[12] In 1915, the capacity of the project was practically equalled but with little or no damage.

A little less than twenty-five years later a flood from the Upper Sacramento River breached the levees in three places and the entire area of Butte City was inundated.[13] Great damage was also done on the Lower Feather River, in California, which the studies showed to be the next most vulnerable area in the Sacramento Valley.

Modifications and improvements in flood-routing methods are made from time to time, and the writer wishes to suggest one possibility which he utilized recently in studies on the Spokane River, in Washington. The authors used a "family of curves" for conditions involving consideration of changing stage (see heading, "Rating Curves"). To substitute a single curve the changing stage factor may sometimes be introduced into the well-known logarithmic gage-discharge relation, thus:

$$Q = A\,[(G - e) + b\,\Delta G]^n \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which A, e, b, and n are constants for any given discharge section and ΔG is the change in stage for the period preceding any observed value of G. The discharge curve is obtained by plotting values of $[(G - e) + b\,\Delta G]$ against

[11] Dean of Eng., Coll. of Eng., Univ. of Wyoming, Laramie, Wyo.

[11a] Received by the Secretary April 7, 1938.

[12] H. R. Doc. No. 191, 73d Cong., 1st Session, p. 55, Zone 2.

[13] *Engineering News-Record*, December 16, 1937.

corresponding discharges. The constants may be found by trial or by the method of least squares. Evidently, when ΔG is very small it may be neglected and the ordinary equation of the discharge curve results.

With a single discharge curve and a single capacity curve, a very easy and rapid graphical-mechanical solution of the storage-discharge equation was suggested in a personal communication to the writer by C. O. Wisler, M. Am. Soc. C. E., which is an improvement on the method published by the writer.[4] By plotting curves for discharge and storage against corresponding elevations, two additional curves for values of $(S + 0)$ and $(S - 0)$ may also be plotted against the same corresponding elevations, as shown in Fig. 10. Elevation scales for

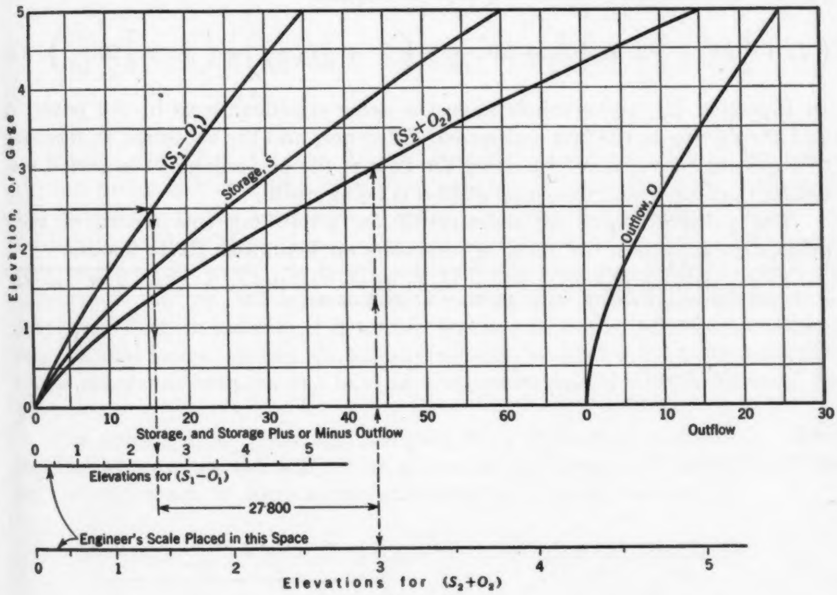

FIG. 10

values of $(S + 0)$ and $(S - 0)$ are then plotted below the curves with a distance apart such that an engineer's scale may be placed between them. To use this method, suppose that a value of 34 000 has been computed for $(S_2 + 0_2)$ at the end of a given period. This value will be found to occur at Elevation 2.5 in Fig. 10 and the corresponding value of $(S_1 - 0_1)$ is 15 800. Now if the grid section of Fig. 10 is enlarged to 10 in., the zero of an engineer's scale (with the same number of divisions per inch as the horizontal scale of the curves) can be placed at 2.5 on the $(S_1 - 0_1)$ elevation scale; then add the value of $I_1 + I_2$ by measuring along the $(S_2 + 0_2)$ elevation scale (say, 27 800), and the elevation at the end of the next period is seen to be 3.0, corresponding to a storage of about 43 600. The engineer's scale is then moved to the right until the zero mark comes to 3.0 on the $(S_1 - 0_1)$ scale, and the operator is ready for the

[4] "Rapid Calculation for Reservoir Discharge," by R. D. Goodrich, *Civil Engineering*, February, 1931, pp. 417, 418.

value of the inflow for the next period. In this manner two computers can do very rapid routing of floods.

If actual flood hydrographs are available for the inflow records, almost any desired degree of accuracy may be obtained by using some shorter period than the 24 hr used by the authors. In some cases, when the flood rise or fall is very rapid, the simple solution given for the storage-discharge relation may not produce sufficiently accurate results. Other solutions of the relationship are possible, as mentioned in the paper.[9] A solution for this case, which would permit the use of curves and methods similar to those used by the authors, can be made quite general by using first and second differences of inflow and outflow rates. The following equation gives one solution,

$$\left(I_1 + \frac{1}{2}\Delta I_1 + \frac{2}{3}\Delta^2 I_0\right)\frac{t}{12} + \Delta 0_0 \frac{t}{18} + \left(S_1 + \frac{1}{6} 0_1 \frac{t}{12}\right) = \left(S_2 + \frac{7}{6} 0_2 \frac{t}{12}\right) \quad \text{..(2)}$$

In Equation (2), the symbols have the same significance as in the paper, Δ and Δ^2 referring to the first and second differences and the subscript, 0, referring to a quantity determined by using the flow at the beginning of the period preceding that for which the computation is being made.

The authors should be commended for publishing this record of their exhaustive studies of the effect of reservoirs on Tennessee River floods.

[9] "The Hydraulics of Flood Movements in Rivers," by Harold A. Thomas, M. Am. Soc. C. E., *Engineering Bulletin*, Carnegie Inst. of Technology, p. 65.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

DESIGN OF PILE FOUNDATIONS

Discussion

By Messrs. Hibbert M. Hill, and Odd Albert

Hibbert M. Hill,[13] Assoc. M. Am. Soc. C. E. (by letter).[13a]—A few years ago the writer had need of a method of analyzing batter-pile foundations and was impressed, in the search for a method, by the little attention writers in English have given this subject. In this respect the author has performed a very worth-while work. It is to be hoped also that the paper will give an impetus to the use of battered piles to resist lateral loads. The writer is convinced that, in general, it is unsafe practice to rely upon vertical piles to resist lateral loads. Never should any reliance be placed upon assumed frictional resistance between the base of a structure and the earth between the piles.

The writer, some years ago, adopted Krey's[14] method of analysis. This method approaches the subject in a manner sufficiently different from the author's approach to make a brief statement of it worth while.

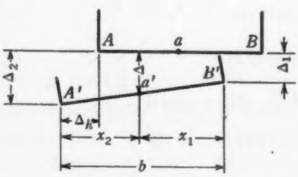

Fig. 10

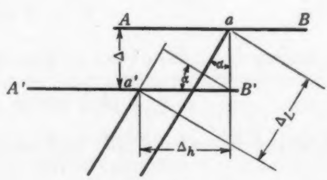

Fig. 11

The assumption is made that the axial deflection of an axially loaded pile is proportional to the applied load, and it is further assumed that the supported structure is rigid, so that the plane of the foundation remains plane after deflection. Consider a foundation, *A-B*, Fig. 10, which, under the action of a given loading, deflects to the position, *A′-B′*. Let *a* be a point on the

Note.—The paper by C. P. Vetter, M. Am. Soc. C. E., was published in February, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[13] Supt., St. Anthony Falls Water Power Co., Minneapolis, Minn.

[13a] Received by the Secretary March 5, 1938.

[14] "Erddurck, Erdwiderstand, und Tragfahigheit des Baugrundes," Berlin, 1932.

foundation, then, Δ = vertical deflection of Point a; Δ_h = horizontal deflection of Point a; and, from the geometry of the figure,

$$\Delta = \Delta_1 \frac{x_2}{b} + \Delta_2 \frac{x_1}{b} \qquad (43)$$

Consider, further, a batter pile in a deflected foundation, Fig. 11: If Δ_h, Δ_1, and Δ_2 are small, and the length of the pile considerable, α, the angle of batter, remains approximately constant. Let the axial deflection of the pile be Δ_L, then,

$$\Delta_L = \Delta \cos \alpha + \Delta_h \sin \alpha \qquad (44)$$

If P is the axial load on the pile,

$$P = k\,\Delta_L \qquad (45)$$

The coefficient, k, may be evaluated by such means as the author uses, or by means of loading tests on piles in the foundation under consideration.

If a computation of deflections is not important and all piles are alike, then the value of k need not be known, as the following equations can be solved directly for $k\,\Delta$. From Equations (43), (44), and (45), P may be evaluated for any pile if the three unknowns, Δ_1, Δ_2, and Δ_h, are determined (from the three conditions of equilibrium). Let V = the resultant vertical load acting on the foundation; and, H = the resultant horizontal load acting on the foundation, at the plane of the bottom of concrete; then:

$$V = \Sigma\, P \cos \alpha \qquad (46a)$$

$$H = \Sigma\, P \sin \alpha \qquad (46b)$$

and (see Fig. 11),

$$M = \Sigma\, x_2\, P \cos \alpha \qquad (46c)$$

Substituting Equation (45) in Equation (46a):

$$V = \Sigma\, k\,\Delta_L \cos \alpha \qquad (47)$$

Substituting Equation (44) in Equation (47),

$$V = \Sigma\,(k\,\Delta \cos^2 \alpha + k\,\Delta_h \sin \alpha \cos \alpha) \qquad (48)$$

Substituting Equation (43) in Equation (48):

$$V = \Sigma \left(k\,\Delta_1 \frac{x_2}{b} \cos^2 \alpha + k\,\Delta_2 \frac{x_1}{b} \cos^2 \alpha + k\,\Delta_h \sin \alpha \cos \alpha \right) \qquad (49)$$

or,

$$\frac{V\,b}{k} = \Delta_1 \cos^2 \alpha\, \Sigma\, x_2 + \Delta_2 \cos^2 \alpha\, \Sigma\, x_1 + n'\, b\, \Delta_h \sin \alpha \cos \alpha \qquad (50)$$

in which n' is the number of piles battered at the angle, α.

Similarly,

$$H = \Sigma\, k\,\Delta_L \sin \alpha = \Sigma\,(k\,\Delta \sin \alpha \cos \alpha + k\,\Delta_h \sin^2 \alpha)$$

$$= \Sigma \left(k\,\Delta_1 \frac{x_2}{b} \sin \alpha \cos \alpha + k\,\Delta_2 \frac{x_1}{b} \sin \alpha \cos \alpha + k\,\Delta_h \sin^2 \alpha \right);$$

or,

$$\frac{H_b}{k} = \Delta_1 \sin \alpha \cos \alpha \, \Sigma \, x_2 + \Delta_2 \sin \alpha \cos \alpha \, \Sigma \, x_1 + n' \, b \, \Delta_h \sin^2 \alpha \ldots (51a)$$

and,

$$\frac{M_b}{k} = \Delta_1 \cos^2 \alpha \, \Sigma \, x_2^2 + \Delta_2 \cos^2 \alpha \, \Sigma \, x_1 \, x_2 + b \, \Delta_h \sin \alpha \cos \alpha \, \Sigma \, x_2 \ldots (51b)$$

From the known foundation dimensions and loads, Equations (50) and (51), may be solved for Δ_1, Δ_2, and Δ_h. Knowing these values, the stress in any pile may be determined from Equations (43), (44), and (45), or the equations may be solved for $k \, \Delta_1$, $k \, \Delta_2$, and $k \, \Delta_h$. By using these quantities in Equations (43), (44), and (45), P may be found directly. This course is only feasible when k is the same for each pile.

There will usually be at least two values of α involved in any batter-pile foundation. The most usual case involves plus and minus values of the same angle. In such a case Equation (50) would be written:

$$\frac{V \, b}{k} = \Delta_1 \cos^2 \alpha \, \Sigma \, x_2 + \Delta_1 \cos^2 \alpha' \, \Sigma \, x_2'$$

$$+ \Delta_2 \cos^2 \alpha \, \Sigma \, x_1 + \Delta_2 \cos^2 \alpha' \, \Sigma \, x_1' \cdots \text{etc.} \ldots \ldots \ldots \ldots (52)$$

and, similarly, for Equations (51). The unprimed values of x_1 and x_2 apply only to piles battered at the angle, α, whereas the primed values apply only to the piles battered at the angle, α'.

Apparently, Krey's method yields results that are somewhat different from those of the author. Applied to Assumption (1) of the author's numerical example, the writer obtains the following comparative values:

Pile No.	Author, Table 2	Writer
1	50.9	41.7
2	2.5	11.7
3	69.5	60.3
4	21.1	30.4

Like other writers on the subject, the author assumes deflections to depend purely upon the elastic properties of the piles. This assumption is tantamount to the assumption that all piles reach a rigid surface at their tips which, of course, is rarely the case. The true pile deflections are a compound of earth settlement, yield of the pile in the soil, and elastic deformation of the soil. Since the deflections upon which the stresses are based are small, and since the relative deflections at the edge and center of a foundation in a compressible soil may be of the same magnitude as, or of a greater magnitude than, the deflections found from the author's or the writer's formulas, the stresses obtained from any formula based upon purely elastic assumptions should be weighed in the light of the probable settlements.

However, analysis such as that given in the paper is a very necessary first approach to the problem (the best now available), and is far superior, say, to the method in which stresses in the batter-piles are first computed as if the piles were vertical, after which the horizontal thrust is based upon the stress thus

computed and the sine of the batter angle. Such methods may lead to dangerous conditions, through a lack of realization of the true action of the inclined piles. The author (see heading, "Two-Dimensional Systems in General"), has pointed out one such possibility. The writer encountered another case in a design having piles battered only in one direction. Under the construction dead load there was danger of marked deformations, if not failure, due to the unbalanced thrust of the batter piles arising from the vertical load, a thrust which could not be balanced until the structure had been completed and backfilled.

The dummy-pile method of accounting for restraint is an ingenious one. The writer's method has been first to make a preliminary computation of deflections, as under the author's Assumption (1), and then on the basis of the deflections thus obtained to approximate the moments and shears in the piles upon the basis of equations such as those of Cummings,[15] or Chang,[16] or Timoshenko.[17] The shear at the base of the structure thus obtained is an additional horizontal force restraining lateral movement, and the moment obtained from these formulas may be added to the moments of the external loads. The pile moments serve to redistribute the external load, often in an important manner. That this occurs in foundations containing only vertical piles, but subject to lateral loads, is not generally realized.

The author has treated only foundations loaded in a single plane. The evaluation of stresses occasioned by eccentricities about both axes of the foundations is possible for many cases. Consider a pier, as shown in Fig. 12(*a*).

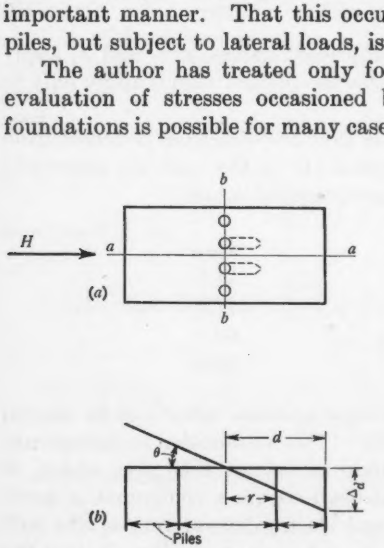

FIG. 12

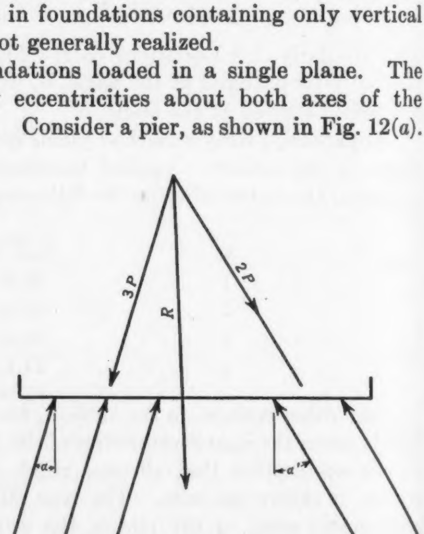

FIG. 13

The pile arrangement is assumed symmetrical about Axis *a-a*. The foundation contains vertical piles, and piles battered parallel to Axis *a-a*. The deflections due to H and V are first evaluated, neglecting eccentricity with respect to Axis *a-a*. Since the piles are symmetrical about Axis *a-a*, the moment in the plane normal to it will cause deflections symmetrical about this axis; thus:

[15] *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 255.
[16] *Loc. cit.*, p. 272.
[17] "Strength of Materials," Vol. II, p. 408.

$M' = \Sigma\, P \cos\theta\, r = \Sigma\, k\, \Delta_L \cos\theta\, r = \Sigma\, k\, \Delta \cos^2\theta\, r = \Sigma\, k\, \Delta_d \frac{r}{d} \cos^2\theta\, r$; hence,

$$\Delta_d = \frac{M'}{\Sigma\, k \frac{r^2}{d}} \cos^2\theta \qquad (53a)$$

and,

$$\Delta = \frac{r}{d} \Delta_d \qquad (53b)$$

If these deflections cause no further deflection in the direction of H, they may be added directly to those previously determined, and from the result, the stresses may be evaluated. In the case shown, the lateral moment causes compression in the batter-piles on one side of Axis *a-a*, and tension, or rather reduction of compressive stress, in batter-piles opposite, of the same amount.

The result is that the horizontal components of the batter-piles are no longer equal on both sides of Axis *a-a*, although H does not change. A couple is formed, therefore, tending to rotate the foundation in the horizontal plane. If this couple is resisted by external restraint as, for example, by adjacent structures, no movement parallel to Axis *a-a* can occur, and the deflections may be evaluated by superposition. It is of interest that the formation of such a couple may be prevented by battering the piles in pairs, one with the load and one against the load, in each row. The horizontal components in each row due to the lateral eccentricity are thereby balanced.

In general, it will be desirable that a foundation utilizing batter-piles contain piles battered in opposition to each other. In the contrary case, the vertical load during construction or at other times when there may be no balancing horizontal load may cause lateral movement of the structure due to the horizontal component of the batter-pile stress. Where piles are battered with, and against, the lateral load, those inclined in the direction of the load may, in the absence of sufficient vertical load on the structure, be subjected to tension—that is, to extractive forces. Although in many instances piles are capable of resisting considerable extractive forces, it is usually not desirable that such forces be permitted to occur, and they will not occur if the vertical load on the foundation is sufficiently large. The effect, then, is to cause a reduction of the compressive load, rather than a tension, and the pile inclined with the load is as effective in lateral resistance as a pile inclined against it.

All the piles in a foundation containing only vertical piles may be loaded equally when subjected to vertical load alone; and rotation in the vertical plane may be prevented, by designing the group so that the centroid of the piles is on the line of action of the resultant load. Such an arrangement will be most economical in its use of piles although not necessarily most economical of superstructure, as all piles will be equally loaded.

Batter-pile foundations may be designed in a similar manner, as the author has indicated. The resultant, R, of the applied loads, excluding the part of the horizontal load taken by the earth in lateral bearing against the piles, Fig. 13, is resolved into two components inclined to the vertical at the adopted angles of batter, α and α'. Each component is divided by the permissible pile load, P, to obtain the number of piles inclined at the respective angles, α and α'. The line

of action of the two components of the resultant is made to act through the centroids of the two-pile groups. All piles will thus be loaded equally and the axial deflection of each pile will be the same. The deflection of the foundation will be vertical and parallel to its unloaded position. There will be a resultant horizontal movement, however, if α and α' are unequal.

It will be noted that the batter angle, α, affects both the number of piles required and their location. These factors, in turn, affect the dimensions of the superstructure. Furthermore, there is usually no reason, other than an economic one, why all piles should be loaded equally, as such cases can be solved by the general formulas, Equations (50), and (51). The principle illustrated by Fig. 13, however, will be found of assistance in determining the most economical over-all design.

ODD ALBERT,[18] Assoc. M. Am. Soc. C. E. (by letter).[18a]—Although interesting and constructive, there is nothing fundamentally new in this paper (as Mr. Vetter infers under the heading, "General Considerations"), except that it may be the first time the subject has been treated in the English language. As indicated in his "Acknowledgments," various pile theories were first developed in Sweden. Professor Per Gullander,[19] of the Chalmers Institute of Technology, Gottenborg, published an article on the subject in 1902. In 1911, R. Ekvall,[20] of the Royal Institute of Technology, Stockholm, added valuable material; and Professor Gullander[21] appeared again in 1912 with his first book on the subject. The previous year, Torsten Hultin, of the Chalmers Institute of Technology, read his paper on "How to Design Pile Foundations," [2] presenting an entirely new graphical method, which has been adopted for general use.

The writer was privileged to study under Professor Gullander, and he remembers well how this famous and beloved teacher modestly stated concerning the various pile theories: "If it requires one week to design a pile foundation by the Ekvall theory, my method will require one day, and the Hultin method will only require one hour." However, he did not mention that this theory could be applied to any combination of pile groups, whereas the Hultin method required at least one axis of symmetry.

The method presented by Mr. Vetter seems to be a combination of the Ekvall and the Gullander methods, with quite a number of important improvements, which will be apparent by comparison with the following review of the original Hultin Pile Theory. The writer has taken the liberty of including a few changes, which he thinks will help to clarify this method.

The Hultin Pile Theory.—The stresses in a pile group supporting a body are caused by a very small movement of this body. This movement, no matter how complicated it may be, can always be divided into two separate movements of the entire body: (1) As a unit in two given directions; and (2) a rotation of the entire body around a selected center.

[18] Asst. Prof. of Structural Eng., Newark Coll. of Eng., Newark, N. J.

[18a] Received by the Secretary March 24, 1938.

[19] "Contributions to the Pile Theory," by P. Gullander, *Teknisk Tidskrift*, 1902.

[20] "The Design of Pile Foundations," by R. Ekvall, 1911.

[21] "The Theory of Pile Foundations," by P. Gullander, 1912.

[2] "Om beräkning ov Grundpälninger," by T. Hultin, Industritidningen, *Norden*, 1911.

Therefore, the loads on the piles will be caused by: (a) The movement of the body in one direction; (b) the movement of the body in another direction; and (c) the rotation of the body around some center.

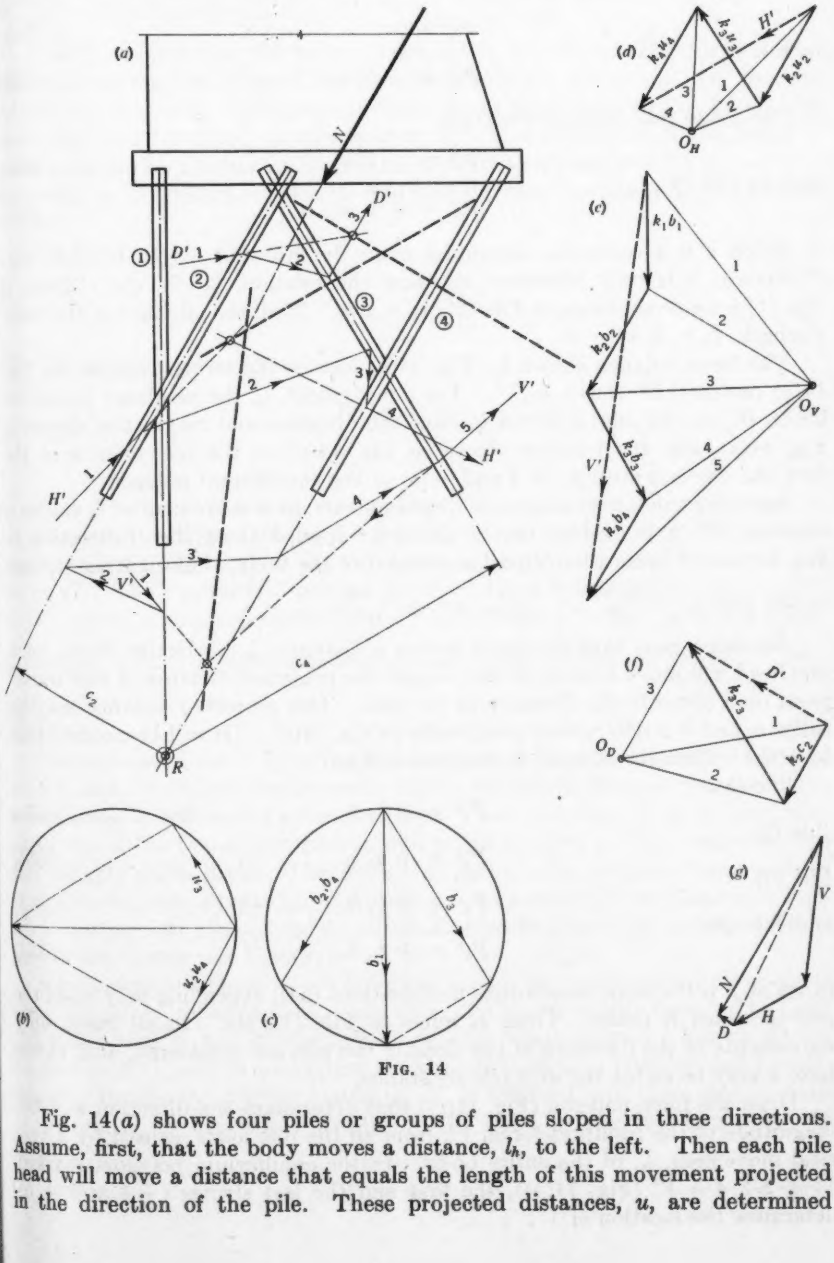

FIG. 14

Fig. 14(a) shows four piles or groups of piles sloped in three directions. Assume, first, that the body moves a distance, l_h, to the left. Then each pile head will move a distance that equals the length of this movement projected in the direction of the pile. These projected distances, u, are determined

graphically in Fig. 14(*b*). It will be noticed that $u_1 = 0$, because Pile (1) is vertical. Then the stresses in the piles, caused by a horizontal movement, l_h, will become:

In Pile (1) (because $u_1 = 0$)

$$P_1^h = 0 \qquad (54a)$$

in Pile (2),

$$P_2^h = + k_2 u_2 \qquad (54b)$$

in Pile (3) (a pull instead of a load),

$$P_3^h = - k_3 u_3 \qquad (54c)$$

and, in Pile (4),

$$P_4^h = + k_4 u_4 \qquad (54d)$$

in which k is a constant, depending upon the size and length of each pile. Therefore, it is only necessary to know the relation, $k_1 : k_2$, etc. Thus, if Pile (1) is twice as strong as Pile (2), $k_1 = 2\,k_2$. If all the piles are of the same strength, $k_1 = k_2 = \cdots k$.

The force polygon shown by Fig. 14(*d*) locates the string polygon in Fig. 14(*a*) (marked H'–1–2–3–4–H'). For a movement, l_h, the resultant to all the loads, H', on the piles is found to have the direction and magnitude shown in Fig. 14(*d*), and the location shown in Fig. 14(*a*) at the intersection of the first and the last strings (+ 1 and − 4) of the equilibrium polygon.

Applying the law of reciprocal displacements, as a movement of l_h causes a reaction, H', it is evident that a unit load applied along the H-direction in Fig. 14(*a*) will cause a horizontal movement of the body, thus creating stresses in the piles of $+ \frac{k_2\, u_2}{H'}$, $- \frac{k_3\, u_3}{H'}$, and $+ \frac{k_4\, u_4}{H'}$.

Assuming next that the body moves a distance, l_v, vertically down, each pile head will move a distance that equals the projected distance of this movement on a plane in the direction of the pile. This projected distance may be called b, and it is determined graphically in Fig. 14(*c*). (It will be noticed that $b_1 = l_v$.) Then the stresses in the piles will be:

Pile (1),

$$P_1^v = + k_1 b_1 \qquad (55a)$$

Pile (2),

$$P_2^v = + k_2 b_2 \qquad (55b)$$

Pile (3),

$$P_3^v = + k_3 b_3 \qquad (55c)$$

and Pile (4),

$$P_4^v = + k_4 b_4 \qquad (55d)$$

in which k is the same constant as in Equations (54), depending only upon the pile to which it refers. Thus, k_1 refers to Pile (1), etc. In all cases, only movements in the direction of the slope of the pile are considered, and, therefore, k may be called the unit pile resistance.

Draw the force polygon (Fig. 14(*e*)) that determines the direction and the magnitude of the resultant force, V', from all the pile loads, caused by a vertical movement, l_v, of the entire body. In the equilibrium polygon, marked V'–1–2–3–4–5–V' (Fig. 14(*a*)), the first and the last strings (+ 1 and − 5) determine the location of V'.

Therefore, a unit load applied along the V-direction in Fig. 14(a) will cause a vertical movement of the entire body, creating stresses in the piles of $+\frac{k_1 b_1}{V'}$; $+\frac{k_2 b_2}{V'}$; $+\frac{k_3 b_3}{V'}$; and $+\frac{k_4 b_4}{V'}$.

Finally, assume that the body rotates to the left around R. As this rotation is considered very small, the movements of the pile-heads may be considered straight, and, therefore, will be in direct proportion to the distance of each pile-head from the rotation center, R. This distance may be called c. As the position of this center is selected at the intersection of Piles (1) and (4), there will be no stresses in Piles (1) and (4) from this rotation. The pile loads will be:

Pile (1),

$$P_1^d = 0 \qquad (56a)$$

Pile (2),

$$P_2^d = + k_2 c_2 \qquad (56b)$$

Pile (3) (rotation causes tension),

$$P_3^d = - k_3 c_3 \qquad (56c)$$

and Pile (4),

$$P_4^d = 0 \qquad (56d)$$

in which k again is the same constant as that used in the other movements.

The force polygon (Fig. 14(f)) gives the direction and magnitude of the resultant force, D', from all the pile loads caused from a rotation around the point, R. The equilibrium polygon, marked D'–1–2–3–D', gives the location of D'. Now, a unit load applied along the D-direction in Fig. 14(a) will create a rotation around R only, creating stresses in the piles, such as $+\frac{k_2 c_2}{D'}$, and $-\frac{k_3 c_3}{D'}$.

Characteristic Lines.—It will easily be seen that the three location-directions, H', V', and D' (called characteristic lines), are independent of the size of the movements, as well as the actual loading. They only depend upon the chosen directions of the two movements as well as the location of the rotation center, and the pile group itself. Therefore, this part can be computed once and for all with complete disregard to the loading. As soon as the characteristic lines are obtained, they can be used for all kinds of loading; and this is the important part in the Hultin pile theory.

Application to a Loading.—A force can always be resolved along three directions; therefore, dissolve the loading along the characteristic lines. The body will then move in two known directions, and rotate around a known center. As soon as H, V, and D are known, the stresses in various piles are easily determined. Hence, the loading resultant, N, is resolved into three components, H, V, and D, as shown in Fig. 14(g). The stress in Pile (2), therefore, will become,

$$f_2^H = + H \frac{k_2 u_2}{H'} \qquad (57a)$$

In this expression, H is an actual load determined from the load, N, carried by the pile group, and given in Fig. 14(g). The ratio, $\frac{k_2\,u_2}{H'}$, can easily be obtained by scaling these distances from Fig. 14(d)—first $k_2\,u_2$ and then H'. Similarly, the load caused by the load, V, on Pile (2) becomes,

$$f_2^V = +\,V\,\frac{k_2\,b_2}{V'} \qquad (57b)$$

and that caused by Load (d) is,

$$f_2^D = +\,D\,\frac{k_2\,c_2}{D'} \qquad (57c)$$

Then the total load on Pile (2) will be the sum of Equations (57), with their signs. Hence,

$$f_2 = +\,H\,\frac{k_2\,u_2}{H'} + V\,\frac{k_2\,b_2}{V'} + D\,\frac{k_2\,c_2}{D'} \qquad (58a)$$

and, similarly,

$$f_1 = 0 + V\,\frac{k_1\,b_1}{V'} + 0 \qquad (58b)$$

$$f_3 = -\,H\,\frac{k_3\,u_3}{H'} + V\,\frac{k_3\,b_3}{V'} - D\,\frac{k_3\,c_3}{D'} \qquad (58c)$$

and,

$$f_4 = +\,H\,\frac{k_4\,u_4}{H'} + V\,\frac{k_4\,b_4}{V'}\,0 \qquad (58d)$$

It will be noted that all complicated computations are eliminated from this method, and that all the stresses can be obtained by simple calculations on the slide-rule.

Mr. Vetter has selected an unusually simple case to demonstrate his method. Its full possibilities, as compared with those of the original (Hultin), cannot be properly appraised until the author has demonstrated its application to the more complicated cases.